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SUMMARY

➤ The triaxial permeability device can be used to evaluate the hydraulic and chemical conductivities of compacted fine grained soils. This device provides control of the primary variables affecting hydraulic and chemical conductivity; void ratio, soil structure, stress history, state of stress, drainage conditions, properties of the permeant, degree of saturation, thixotropy, and gradient.

Methodology for evaluation of conductivity for fine grained soils using water and/or chemical permeants is discussed. The design of the permeability apparatus is discussed in detail followed by descriptions of the procedures used to prepare and consolidate specimens, monitor the equilibrium hydraulic conductivity, change permeants, and interpret the data. A limited amount of initial test data is presented and discussed, along with recommendations for continued study. (These)



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TRIAXIAL PERMEABILITY DEVICE

A THESIS
Presented to
The Faculty of the Division of Graduate Studies

By

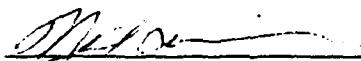
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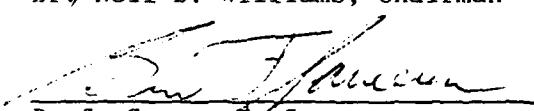
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of the Requirements for the Degree
Master of Science in Civil Engineering

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February, 1987

TRIAXIAL PERMEABILITY DEVICE

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The fabrication of the Triaxial Permeability Device required many hours of work from the Geotechnical Staff. Our machinist, Seph Scott, provided the largest effort in terms of expertise and hours. Also, the technical and moral support of Ken Thomas is also appreciated; for his knowledge and efforts helped overcome many difficulties encountered in the lab. In addition, my thanks to Vicki Clopton who assisted me with the preparation of this text.

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SUMMARY

The triaxial permeability device can be used to evaluate the hydraulic and chemical conductivities of compacted fine grained soils. This device provides control of the primary variables affecting hydraulic and chemical conductivity; void ratio, soil structure, stress history, state of stress, drainage conditions, properties of the permeant, degree of saturation, thixotropy, and gradient.

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CHAPTER I

INTRODUCTION

The design of subsurface storage facilities for hazardous wastes requires accurate evaluation of the conductivity of fine-grained soils for the permeants expected to be in contact with the compacted clay liner. A variety of test methods have been presented for performing such testing (Anderson and Brown, 1981; Olson and Daniel, 1981).

The development of testing equipment and procedures to accurately evaluate the conductivity of compacted fine grained soils requires careful monitoring and control of the variables which affect the conductivity. These variables include void ratio or porosity, soil structure, state of stress, stress history, properties of the permeant, degree of saturation, thixotropy, and gradient. Each variable will be discussed further in this paper. The Triaxial Permeability Device described herein provides the maximum degree of flexibility and control of the parameters which impact conductivity.

CHAPTER II

TERMINOLOGY

Hydraulic Conductivity and Permeability

The Darcy equation, published in 1856, may be used to evaluate flow through a porous media. The equation, based on the experimental observations, is as follows:

$$q = -kia \quad (1)$$

where q is the flowrate (L^3/T); i is the gradient (-); A is the cross sectional area of the flowpath (L^2), and k is a proportionality constant (L/T), called the coefficient of permeability or hydraulic conductivity. The hydraulic conductivity is both a function of the properties of the fluid and the porous medium. For the purpose of this article, when water is the permeant the proportionality constant shall be called the hydraulic conductivity. When permeants other than water are used, the proportionality constant shall be called the chemical conductivity.

Another, more fundamental form of the Darcy equation is:

$$q = - \frac{KigA\phi}{\mu} \quad (2)$$

where K is the intrinsic or specific permeability (L^2) and is a function only of the properties of the porous media; ρ is the density of the pore fluid (M/L^3); μ is the dynamic viscosity of the pore fluid (M/LT); and g is the acceleration due to gravity (L/T^2) (Olson and Daniel, 1981).

Combining equations (1) and (2):

$$k = \frac{Kg\rho}{\mu} \quad (3)$$

Therefore, the hydraulic conductivity is a function of the permeability of the porous media.

Hydraulic conductivity is generally used to describe the rate of flow of water through a porous media. Intrinsic permeability is used to describe the effects of other permeants on hydraulic conductivity. The relationship between hydraulic conductivity and intrinsic permeability can be used to evaluate the changes in the flow characteristics of a porous media permeated by various fluids.

CHAPTER III

FACTORS AFFECTING THE HYDRAULIC OR CHEMICAL CONDUCTIVITY

There are many variables that affect the hydraulic or chemical conductivity of soils. This discussion will be limited to those parameters that have the most significant impact on hydraulic or chemical conductivity. These are the void ratio or porosity, soil structure, state of stress or stress history, properties of the permeant, degree of saturation, thixotropy, and gradient. The affects of the double layer and cation type on hydraulic conductivity are also evaluated within the discussion on properties of the permeant.

Void Ratio

The void ratio, e , is the ratio of the volume of the voids to the volume of solids. The relationship between void ratio and hydraulic or chemical conductivity is not unique, due to the variability of soil structure. The soil structure, as discussed in the next section, dictates the tortuosity of the flow path through a soil.

The hydraulic or chemical conductivity analyses are often reported as a function of the void ratio of the soil. For a particular soil prepared in a similar manner the

hydraulic or chemical conductivity typically decreases with decreasing void ratio. This is due primarily to a reduction in the size of the flow paths. However, two soil specimens prepared from the same soil type at equal void ratios may have different conductivities. This may be due to differences in soil structure between the two specimens.

Similarly, two different soils at the same void ratios may have different conductivities. This can be due to differences in the soil structure and differences in the soils mineralogy.

Therefore, it is not necessarily the void ratio that governs conductivity, rather it is the distribution of the voids that are of importance. This distribution of voids, or tortuosity, is the result of soil structure, as discussed in the next section.

Soil Structure

The structure of the soil particles within a soil mass is an important factor in assessing the rate of fluid flow through a soil. The structure of the soil particles varies depending upon compaction methods, water content, pore fluid, state of stress, and stress history.

Compaction methods for clay liner systems may be divided into either static or kneading methods. The soil structure is affected by the method of compaction, the compactive effort, and the water content of the soil.

If a soil mass is made up of flocculated clay particles that are dry of optimum, both kneading and static compaction will cause the soil to be more dense, but will generally not disperse the flocs. This is because the compactive effort cannot overcome the interparticle attractive forces.

As the soil approaches the optimum water content, charge deficiencies are satisfied, resulting in particle orientation under the influence of the compaction. Static compaction of a soil wet of optimum will yield a dispersed soil structure. If kneading compaction is used, particles orient themselves in the direction of the shear plane caused by the compaction. This leads to a more dispersed soil structure on a micro scale, but a more random structure on a macro scale (Lambe, 1958).

A flocculated soil generally has larger flow paths for fluid flow and a lesser degree of tortuosity than a dispersed soil, resulting in higher conductivity for a soil with a flocculated structure at the same void ratio. Since low conductivity is the objective of clay liners, steps must be taken to ensure a dispersed structure is obtained.

The pore fluid in the soil during compaction is also important to the soil structure. Furthermore, a change in the pore fluid of a soil may lead to changes in structure. In general, since most fluids have dielectric constants less than that of water, changing the pore fluid of a soil from water to another fluid results in a decrease of the double

layer thickness. Further details on this phenomena are discussed in the section titled "Volume Change as a Function of the Permeant and Boundary Stresses".

State of Stress

Effective Stress Concept

The effective stress concept, first defined by Terzaghi in the 1920's, states effective stress is equal to total stress minus the pore pressure. Effective stress controls certain aspects of soil behavior, most notably compression or consolidation, and strength (Lambe and Whitman, 1969). Compression in soils causes changes in the void ratio, and the void ratio of a soil is one factor that affects conductivity. Zimmie et al (1981) reported that increasing the effective stress on a soil causes a decrease in hydraulic conductivity, and concluded that hydraulic conductivity and void ratio typically vary as a function of the log of effective stress.

Because of this relationship between effective stress and conductivity, it is recommended that soil be tested at the same stress level the soil would be subjected to in-situ. Where this is not practical, the relationship between effective stress and conductivity should be established.

Volume Change as a Function of the Permeant and Boundary Stresses

The chemical properties of a permeant may effect the double layer thickness of a fine grained soil, resulting in volume change. This volume change may begin as soon as the soil is exposed to the chemical permeant, and the affected area will expand as the permeant spreads through the soil under a gradient. The properties of the permeant alter the clays double layer, and the subsequent volume change is a function of the boundary stresses.

The double layer thickness is primarily a function of soil type, surface charge density or surface potential of the clay, electrolyte concentration, cation valence, dielectric constant of the medium, and temperature (Mitchell 1976). The amount of volume change, which is related to the change in conductivity, is a function of the state of stress and changes in the double layer thickness. The changes in double layer thickness can increase or decrease the diameter of the flow channels through the soil mass. Since hydraulic conductivity is directly proportional to the second power of diameter of the flow channel (Mesri and Olson, 1971), and double layer thickness effects the flow channel diameter, then alteration of the pore water chemistry may affect hydraulic conductivity. This alteration of pore water chemistry may cause changes in the net attractive or repulsive forces of the clay particles.

Increasing the attractive forces may increase flocculation, while increasing repulsive forces may cause dispersion in a sedimented soil specimen. Because of the relationship between flow channel diameter and hydraulic conductivity, a flocculated soil fabric would typically have a greater hydraulic conductivity than a dispersed soil fabric (for the same soil and the same void ratio and effective stress).

Of the variables that effect double layer thickness, surface charge density or surface potential is a function of the clay mineral, and temperature is a function of local environmental conditions. Since triaxial permeability testing does not initially change the soil or the temperature, the introduction of chemical permeants in testing only effects the dielectric constant, electrolyte concentration, and cation valence of the pore fluid.

The dielectric constant of a fluid is a measure of the ease with which molecules can be polarized and oriented in an electric field (Mitchell, 1976). For instance, if two charged particles exist at a fixed distance apart in a vacuum, they have an attraction or repulsion for each other. If water, whose dielectric constant is around 80 at 20 degrees Celsius, was placed between those same two fixed particles, the attractive/repulsive forces would be 1/80 th of what they are in the vacuum (Toon et al, 1968). The dielectric constant of a fluid may be thought of as its

ability to insulate electrical charges.

The thickness of the double layer, $1/K$, is approximately calculated by (Mitchell, 1976):

$$\frac{1}{K} = \left(\frac{DkT}{8\pi v^2 \eta_0 e^2} \right)^{\frac{1}{2}} \quad (4)$$

where: D = dielectric constant of the fluid

k = Boltzmann constant = $1.38 \times 10E-16$ erg/ $^{\circ}$ K

T = temperature in degrees Kelvin

η_0 = reference ion concentration = ions/cm 3

e = unit electronic charge = $1.6 \times 10E-20$ Coulombs

v = ionic valence

Therefore, double layer thickness increases with the square root of dielectric constant, and since most fluids have dielectric constants lower than that of water, permeating a different fluid into a clay water system will normally cause a decrease in double layer thickness, with all other factors in equation five remaining constant.

The other factors that will change double layer thickness are ion concentration and ionic valence. Double layer thickness will decrease inversely as the valence and the square root of the concentration increases, all other factors being constant. As indicated by equation 4, changing the pore fluid in clay will likely cause changes in double layer thickness and subsequently cause volume change in the soil.

The change in chemical conductivity of a soil due to variations in double layer thickness is primarily a function of the state of stress. For example, if a soil is permeated with a chemical which increases the double layer thickness, the chemical conductivity may increase, decrease or stay the same depending on the state of stress. If the soil is unconfined, the soil matrix will likely swell, resulting in an increase in porosity and little or no change in chemical conductivity. If the soil is unconfined and a relatively high gradient is induced during analysis, the chemical conductivity may increase or decrease, depending on whether particle migration causes clogging or piping. If the soil specimen is confined, the expansion of the double layer will generally result in a decrease in the pore diameter and a decrease in the chemical conductivity.

When conditions are such that the double layer thickness decreases, a soil may undergo shrinkage. Again, this is a function of boundary stress or overburden pressure. Under low boundary stresses, a soil may decrease in volume until double layer repulsion and/or friction between the particles stops the shrinkage. Non-linear attractive forces induced by closer particle spacing may cause shrinkage cracks. Under higher stress, the shrinkage may be greater because the confining stress could overcome particle repulsion and friction causing reorientation and densification of the soil particles. Therefore, decreases in double

layer thickness may also reduce, increase, or have no effect on the chemical conductivity, depending primarily on the state of stress.

An example of how overburden pressure influences chemical conductivity is presented by Foreman and Daniel, 1986. A permeability test was performed in a compaction mold permeameter, using methanol as the permeant in a kaolinite specimen. After two pore volumes of methanol had passed through the specimen the effective stress was increased and another trial was performed. This procedure was repeated through a series of analyses. A similar permeability test was performed in the same apparatus with the same type of soil using water as the permeant. The hydraulic conductivity remained constant over the range of stresses. At the lower effective stress levels the soil had a higher conductivity for methanol than water. As the stress increments increased, the chemical conductivity decreased, eventually becoming less than the hydraulic conductivity at the same stress level.

In this case the methanol decreased the double layer thickness of the kaolinite, causing the formation of macropores and allowing methanol to flow through the newly formed voids. As the overburden increased, the soil consolidated into a denser configuration. Therefore, the particles had a more intimate particle arrangement as compared to the sample tested with water, thereby increasing

the tortuosity and reducing the conductivity.

Stress History

The stress history of a compacted soil relates the maximum compactive stress that the soil has ever been subjected to σ_c , to the present stress level σ_o . This can effect the soil strength, compressibility, conductivity, interparticle attractions, and fabric (Lambe and Whitman, 1969).

If a compacted soil is overconsolidated, $\sigma_c/\sigma_o > 1$, then it has a lower void ratio and water content than if it was normally consolidated (Lambe and Whitman, 1969). The lower void ratio results in a decreased diameter of flow paths and increased tortuosity, thereby decreasing the conductivity of the intact soil mass. The flow path diameter and tortuosity vary with the soil fabric.

Conversely, some affects of stress history can cause increases in hydraulic conductivity. A soil mass that contains failure planes or slickenslides from past shear stresses, stress relief cracks, or desiccation cracks has potential flow paths for liquids. Along a shear plane the soil particles are oriented parallel to the plane resulting in a less tortuous path for fluid flow as compared to the intact soil mass. Stress relief or desiccation cracks provide openings within a soil mass that may allow fluid flow.

Structural features of undisturbed in-situ soils may control the direction and magnitude of flow through certain types of undisturbed clay soils. Since these features typically repeat on a relatively large scale, it may not be possible to accurately measure the conductivities of undisturbed clay soils using laboratory analyses. However, in most clay liners the soil is recompacted, which tends to remove the structural features affecting conductivity. Therefore, laboratory analyses would be expected to provide accurate measurements for the conductivities of compacted clay soils where the structure of the clay is preserved during the analysis.

Degree of Saturation

The degree of saturation, S, is defined as the volume of water divided by the volume of the voids in a soil (typically reported as a percentage). The degree of saturation in a soil specimen has a direct effect on hydraulic conductivity. Water does not flow through air bubbles, therefore, air reduces the void space water may occupy in a soil (Olson and Daniel, 1981). As the degree of saturation decreases, hydraulic conductivity decreases. Mitchell et al (1965) reported hydraulic conductivity increased as the degree of saturation increased in a silty clay from 90 to 95 percent. The hydraulic conductivity of the 90% saturated specimen was 1.5×10^{-6} cm/sec, whereas

the hydraulic conductivity of the 95% saturated specimen was 2×10^{-6} cm/sec. Borden and Sides, 1970, found the coefficient of hydraulic conductivity varied by 60 to 100% as a function of the saturation level. Other research (Christianson, 1944; Burmister, 1954) has shown changes in conductivity from 2 to 40 times, depending on the degree of saturation. It should also be remembered that one of the assumptions of the Darcy Equation is complete saturation of the soil.

Because of the effect of saturation on hydraulic conductivity, it is necessary to saturate all samples before conductivity testing. Not only does this provide conservative results, but it also gives a baseline value of saturation to start each analysis from.

Thixotropy

Thixotropy is defined as the ability of a soil mass to gain strength with time. The process of laboratory soil testing always causes some degree of sample disturbance, and some reorientation of soil particles. After sample preparation, the soil will tend to become more flocculated with time (Dunn, 1983). This increased flocculation can cause an increase in flow path diameter, increasing hydraulic conductivity.

Mitchell et al (1965), tested a silty clay for the effects of thixotropy on hydraulic conductivity. In

comparing the conductivity of a specimen tested immediately after preparation and a specimen tested 21 days after preparation, the 21 day old specimens always had greater conductivities.

Depending on the molding water content, the differences caused by thixotropy could be as great as an order of magnitude. Dunn (1983), found that Altamont soil tested after aging 15 days had a conductivity 5.6 times greater than a sample tested immediately after preparation.

Based on this information, a conclusion can be made that hydraulic conductivity testing yields unconservative results if thixotropic results are not accounted for. Since the purpose of conductivity analyses is to evaluate the long term conductivity of the clay liner, the soil specimens used in the analyses should be prepared sufficiently in advance of the analyses that thixotropy would have only minimal affects on conductivity. This may require testing many identical specimens at various ages in order to determine when conductivity undergoes minimal changes with further increases in time.

Gradient

Gradient may be defined as the difference in head between two points in a porous media, divided by the distance between those two points. In ground-water flow, gradients are usually low and have little immediate effect

on soil structure. However, in laboratory testing, gradients are often increased to reduce testing time. These high gradients may dramatically influence conductivity.

One way high gradients affect soil structure is through consolidation of the sample. When excessive gradients are used, consolidation may be observed through axial deformation (compression). When fluid flows through an isotropic soil, a seepage force is exerted in the direction of flow. Some of this seepage force is transferred to the soil skeleton by frictional drag, and is related to head loss (Lambe and Whitman, 1969). The seepage force per unit volume of soil (J) may be expressed as:

$$J = i\tau_w \quad (5)$$

Where i is the gradient and τ_w is the unit weight of water. This equation shows that increasing gradient causes an increase in the seepage force. Through frictional drag the seepage force can cause consolidation of the soil skeleton. This consolidation can squeeze out pore water, and cause the flow into a specimen not to equal outflow. Therefore, if inflow does not equal outflow, or if axial strain is observed during conductivity testing, then the gradient may be causing the specimen to consolidate. This causes internal changes in particle arrangement or particle migration. If inflow is equal to outflow, and axial strain is equal to zero, then internal changes in particle

arrangement or particle migration affects are assumed to be negligible. However, if conductivity values are not consistent, then these affects are not assumed to be negligible.

The Darcy Equation (eq. 1) indicates a linear relationship between gradient and flowrate, and assumes laminar flow. This is not necessarily true because high gradients may cause localized turbulent flow and changes in structure in sandy soils. This change in soil structure can either increase or decrease conductivity. A decrease in conductivity may be caused by particle migration (Mitchell and Younger, 1967) or specimen consolidation under gradient induced seepage forces. Particle migration may cause clogging of the pores or development of preferential flow paths. Conversely, nonlinear increases in conductivity may be caused by piping in the sample or erosion around the sample.

Previous study of the maximum allowable gradient has not produced definitive guidelines to follow. Zimmie et al (1981) state that gradient associated soil disturbance rarely occurs with gradients below about 20. They recommend keeping the gradient in the range of 5-20, preferably using those in the low end of that range. Dunn (1983), found conductivity decreased as gradient increased from 20 to 200.

Foreman and Daniel, 1986, evaluated the influence of soil type and gradient on conductivity. They performed

hydraulic conductivity tests on three soil types at various gradients in a flexible walled permeameter. Variation of gradient from 10 to 300 in kaolinite produced little change in the hydraulic conductivity, while the Hoytville and Lufkin clays both showed decreases in conductivity with increasing gradient. These decreases were attributed to the closing of macropores due to the higher effective stresses associated with high gradients and particle migration.

The maximum allowable gradient is literally a function of soil type as well as effective stress, permeant type and other variables. Therefore, the maximum allowable gradient should be evaluated for each analysis or group of analyses. As in all soil testing, it is most desirable to test soils in a manner that most closely duplicates the expected field conditions. If this is not practical, then the variation from field conditions must be evaluated for the effect on test results.

CHAPTER IV

PERMEAMETERS

The hydraulic or chemical conductivity of soils has been evaluated using a variety of permeability testing equipment. The most widely used permeameter cells are the consolidation cell, compaction mold, column cell, and triaxial cell (Dunn, 1983). Hydraulic conductivity may not be greatly influenced by permeameter type (Mitchell et al, 1965; Foreman and Daniel, 1986), however chemical conductivity is influenced by permeameter type (Foreman and Daniel, 1986)..

Consolidation Cell

The consolidation cell (ASTM, 1970) may be used as a fixed walled permeameter (fig. 4.1). Specimens analyzed in this device may be undisturbed or remolded. Preparation and mounting of the specimen must be performed carefully to minimize voids between the specimen and the cell wall. Leakage along the sides could cause unrealistically high values of conductivity, especially in soils that shrink when exposed to a permeant. Zimmie et al (1981) recommends filling the annular space between the sample and cell wall with a sand bentonite mixture to prevent side flow along the specimen. However, it is difficult to prove that no side

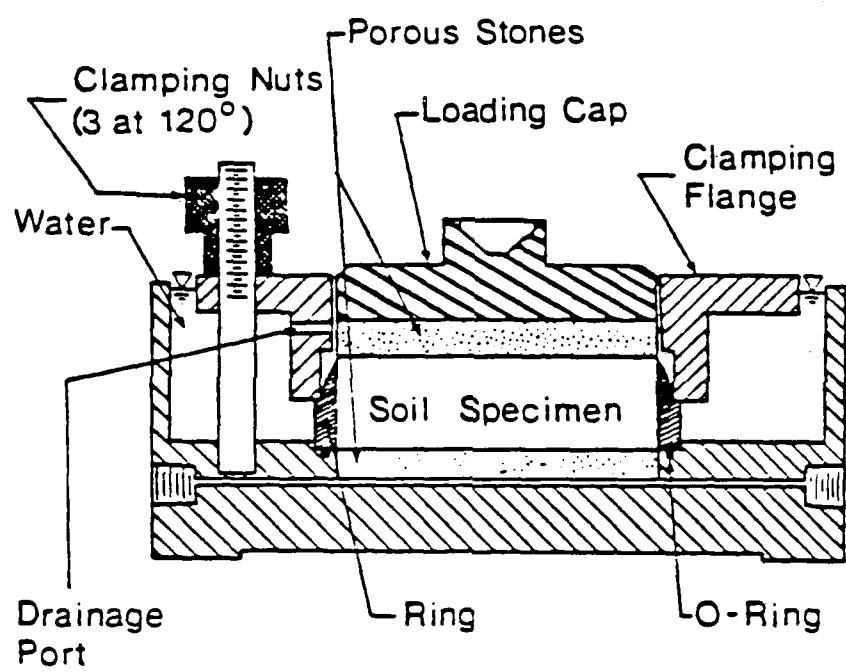


Fig. 4.1 Consolidation Cell Permeameter (Olson et al, 1981)

flow would occur under these conditions, and the sand-bentonite mixture may participate in chemical reaction. Olson and Daniel, 1981, were able to measure conductivities less than $1 \times 10E-12$ cm/sec using consolidation cells as permeameters. Successful analyses can be performed with this cell, however soil and permeant properties must be considered, especially with regard to potential specimen shrinkage.

The primary disadvantage of the consolidation cell permeameter is that it does not allow for complete control of stress conditions. The specimen is loaded axially with no lateral strain. This limitation prevents complete control over the lateral stresses. Test procedures using consolidation cell permeameters are outlined by the Corps of Engineers (1980), and ASTM (1970).

Compaction Molds

Compaction molds (fig. 4.2) have also been used as fixed walled permeameters. Analyses performed in compaction molds are quite similar to the consolidation cell, and have the same limitations. These limitations include the possibility of voids occurring between the soil specimen and cell walls, and a lack of control over the state of stress. Test procedures using compaction molds as permeameters are presented by USBR (1974), ASTM (1979), and Corps of Engineers (1980).

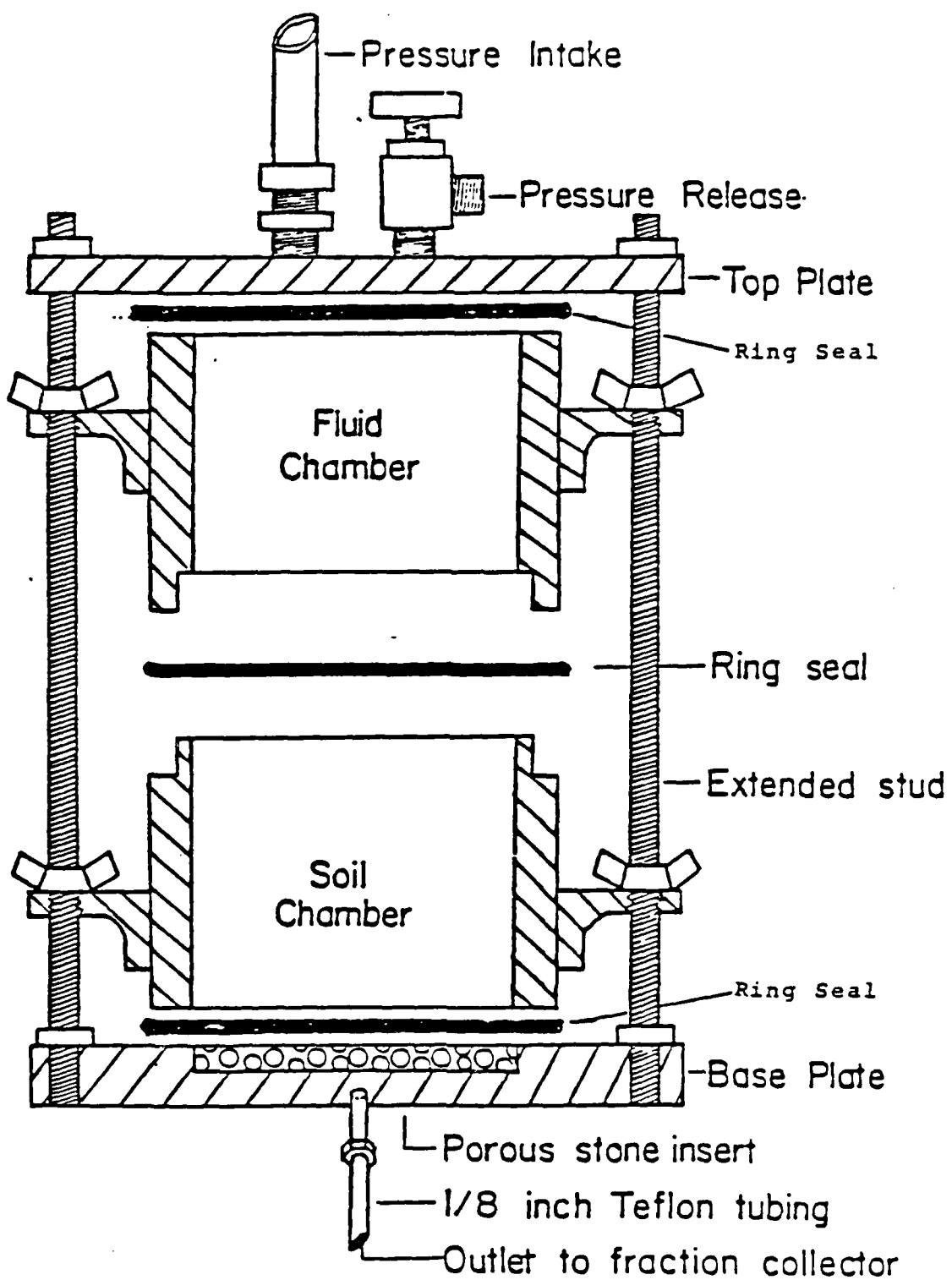


Fig. 4.2 Compaction Mold Permeameter (Matrecon, 1980)

Column Cells

Column cells (fig. 4.3) are thick walled glass columns that allow for vertical flow through a soil specimen. The device is very similar to other fixed walled permeameters since there is no control of lateral stresses and side leakage around the sample. Additional limitations include the lack of vertical effective stress, the lack of control of changes in the void ratio, and the poor control over soil placement. Green, Lee, and Jones (1981) evaluated the effects of several chemical permeants on shale using a glass walled column cell permeameter. They noted certain solvents cause shrinkage in soils and lead to the formation of channels and cracks, allowing permeant flow that bypassed the intact soil mass. This shrinkage could cause the soil to pull away from the sides of a fixed walled permeameter causing flow around the sample, otherwise known as "short circuiting".

Triaxial Cell

Triaxial cells have also been used as permeameters for evaluation of conductivity. Bishop and Henkel (1962) outlined procedures for hydraulic conductivity testing in a triaxial cell. In the triaxial cell, field stresses, pore pressures, and soil conditions may be reproduced. Since this cell uses a flexible membrane to encapsulate the sample, cell pressure against the membrane provides complete

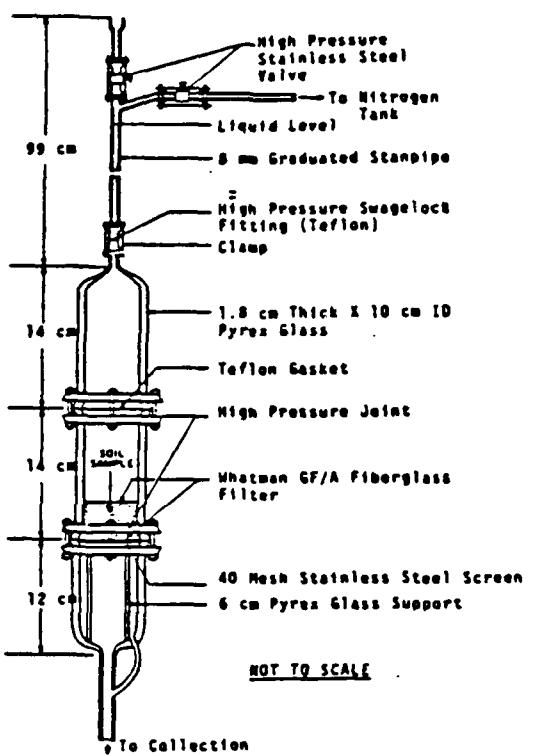


Fig. 4.3 Column Cell Permeameter (Green et al., 1981)

contact between the membrane and the specimen. This eliminates the problems of side leakage encountered with fixed walled permeameters. Lateral stresses, controlled by cell pressure, can more realistically model field conditions. This, coupled with independent axial loading, allows for modeling of anisotropic stress conditions.

The effective stress level (chapter III) greatly affects a soils hydraulic conductivity (Zimmie et al, 1981). Therefore, evaluation of conductivity in the laboratory requires control of the state of stress in the soil specimen. Generally, the triaxial cell is recognized as state-of-the-art for duplicating in-situ soil stresses.

The triaxial cell allows for control of stresses on the sample, backpressure saturation, and most importantly, has a flexible membrane around the sample. Having the soil enclosed in a flexible membrane is important for many reasons. First of all, it allows the soil sample to be subjected to the desired cell pressure. Also, it will expand or contract as the sample undergoes lateral strain and volume change. This is important since it prevents gaps from forming between the sample and the membrane. Such gaps can provide a preferred path around the soil specimen and give unrealistically high values for conductivity. Finally, if a plexiglass triaxial cell cylinder is used then the behavior of the specimen may be observed during testing.

Comparison of Cells

Comparisons between fixed walled permeameters and flexible walled permeameters have shown both agreement and disagreement. Mitchell, Hooper, and Campanella (1965) tested a silty clay for hydraulic conductivity in both a triaxial cell and lucite walled fixed ring permeameter. Their results showed very minor differences between the values of hydraulic conductivity of the silty clay measured in both cells. Green et al (1981) noted certain solvents caused shrinkage of soil in a glass walled column test, leading to short circuiting of the permeant around the sample.

Foreman and Daniel (1986) performed hydraulic and chemical conductivity tests on three different soils using a consolidation cell, compaction mold, and flexible walled permeameter. For hydraulic conductivity testing, similar values for hydraulic conductivity were obtained, regardless of the type of permeameter. In their chemical conductivity test, the compaction mold permeameter gave higher conductivities than the flexible walled permeameter. This was due primarily to the effect of sidewall leakage in the fixed walled permeameter, which was prevented when the flexible walled permeameter was used.

Since the triaxial device more closely models field conditions and reduces the potential for side leakage or short circuiting, this research recommends the use of a triaxial permeability device for the evaluation of the hydraulic and chemical conductivity of soils. The device provides maximum flexibility of the variables affecting the conductivity and ensures the field conditions are adequately modeled.

CHAPTER V

TRIAXIAL PERMEABILITY DEVICE

Triaxial Cell

The use of a triaxial cell for hydraulic conductivity testing has been presented previously by Bishop and Henkel (1962), Dunn et al (1984), as well as many others. The following original design for the Triaxial Permeability Device incorporates some of their ideas with many modifications to yield a design that provides maximum control of the variables affecting hydraulic conductivity.

The Triaxial Permeability Device consists of a triaxial cell and control panel, as drawn in figure 5.1. The triaxial cell is composed of top and bottom sample platens, top and bottom cell plates, rod guide, load rod, and cylinder. The top and bottom sample platens, top and bottom cell plates, load rod, rod guide, and all tubing fittings are made of type 304 stainless steel to allow for maximum resistance to degradation by the various permeants. The effluent tubing is composed of teflon to provide chemical resistance and flexibility. All fittings that enter into the cell are made pressure tight through the use of O-rings or stainless steel ferrules. The O-rings may be made of Viton, neoprene, or any other elastomer depending on the

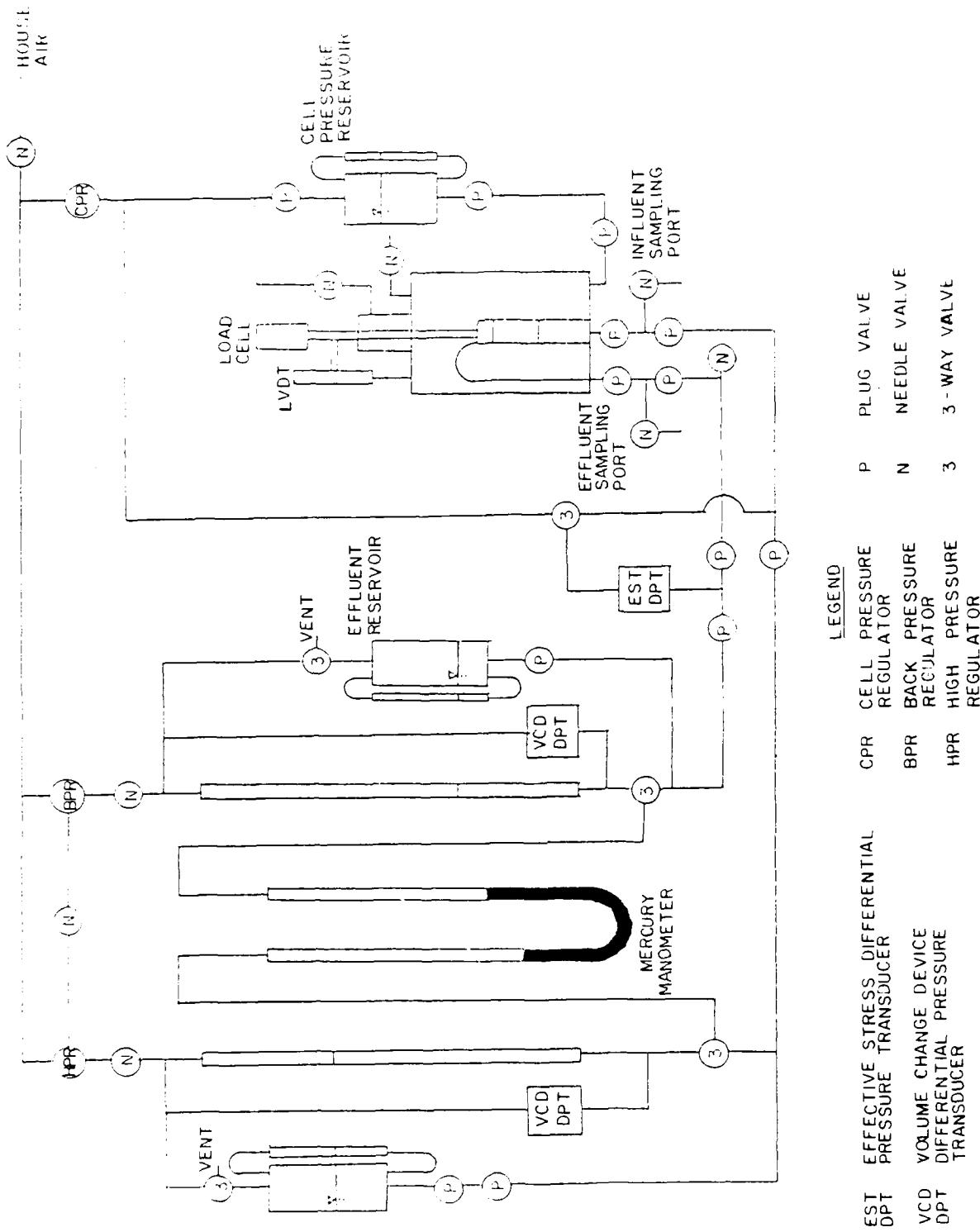


Fig. 5.1 Schematic, Triaxial Permeability Device

type of permeant. The cylinder may be either plexiglass or stainless steel, depending on the composition of the cell fluid and the level of confining stress.

Control Panel

The control panel controls or monitors the application of cell pressure, backpressure, high and low gradient flow, the measurement of total and effective stresses, and volume flow in and out of the specimen. Digital readouts of stress conditions and volume changes can be made through the use of differential pressure transducers.

Volume Change and Effective Stress Transducers

In order to remotely monitor variations in inflow and outflow volume, changes in fluid density and the total and effective stresses, differential pressure transducers are used in this system to monitor the volume changes and stresses in the soil specimen.

Differential pressure transducers are electronic devices used to measure the difference in pressure between two points in a system. If it is desireable to know the difference in pressure between two lines, be they fluid or gas, each line is attached to appropriate connection points on these transducers. A diaphragm inside the transducer deflects in proportion to the difference in pressure. The deflection of the diaphragm is measured with strain gages via a Wheatstone Bridge. The output from the bridge is

linearly proportional to the difference in pressure across the diaphragm.

Two volume change differential pressure transducers are used, one of which is located at the inflow standpipe, and the other at the outflow standpipe. The inflow volume change transducer may be correlated with the height of fluid in the inflow standpipe. As fluid flows into the sample, the pressure difference across the transducer decreases.

The volume change transducers are calibrated by allowing a given volume of fluid of known density to flow out of the inflow standpipe, and the corresponding change in the digital output is recorded. A relationship between the change in height of fluid in the inflow standpipe and the output of the transducer may be correlated with the volume of the influent.

The outflow volume change differential pressure transducer measures the difference in pressure between the top and bottom of the outflow standpipe. This measures the volume of flow leaving the soil sample. The transducer operates the same way as the inflow transducer.

The effective stress differential pressure transducer measures either the effective stress in the soil specimen or the difference in pressure head across the specimen. To measure effective stress one side of the transducer is connected to the cell pressure line and the other side to the backpressure line. The readout from this configuration

is total stress minus pore pressure which equals effective stress.

By using a three way valve located near this transducer, the inflow head can be measured, which gives the operator the ability to measure change in head across the specimen. This is necessary to evaluate gradient when the high gradient system is used.

Additional information regarding the mechanical operation and procedures for these transducers in measuring volume changes and pressures is presented in chapter 6, entitled "Methodology".

Pressure Regulators

The pressure to the board is supplied by filtered house air, and controlled by the cell pressure, backpressure, and high pressure regulators. The cell pressure regulator controls air pressure to the cell recharge reservoir and the triaxial cell. The cell pressure provides confining pressure and ensures the membrane has intimate contact with the soil specimen. The backpressure regulator controls the amount of backpressure (or pore pressure) to the specimen and thus the effective stress. The high pressure regulator is used to control the amount of pressure on the influent line for high gradient testing.

Reservoirs

The Triaxial Permeability Device uses three fluid reservoirs. The influent reservoir stores permeant to recharge the hydraulic tubes and set the gradient. The effluent reservoir stores the effluent after its volume has been measured in the outflow standpipe. The main cell recharge reservoir is used to supply the triaxial cell with confining fluid in response to the volume change of the specimen.

Gradient Control

The gradient may be applied to the specimen using one of three systems: the low gradient open loop system; the closed loop system; or the high gradient open loop system.

The low gradient open loop system is typically used when the maximum desired gradient is less than 20. Gravitational forces on the influent result in flow through the sample at gradients less than 20. This gradient is measured using the volume change differential pressure transducers or by reading the height of the permeant on the inflow and outflow standpipes. A low gradient test may be performed using the open or closed loop system. For gradients below 20, the open system is used. If this system is used on a specimen with low conductivity, then testing time could become unreasonably long. If time is not a factor, then consideration must be given to the amount of

evaporation that takes place on the surface of the inflow and outflow standpipes. The rate of evaporation could exceed the rate of flow through the specimen, yielding unacceptable error. If evaporation is going to be a problem, then the closed loop system should be used.

The closed loop system is used for gradients less than 80. The gradient on this system is measured by reading the difference in height between the two columns of mercury in the manometer. This system does not allow air into the fluid lines, and therefore minimizes evaporation. For that reason, this system is most desireable for soils with low hydraulic conductivities. This system does not alleviate the problem of long testing times for fine grained soils.

The gradient for the high gradient open loop system ($i > 80$) is controlled by applying air pressure through the high air pressure regulator. This pressure is applied to the column of fluid in the inflow standpipe. The gradient is measured by reading the effective stress transducer output.

In order to eliminate excessive testing times, the open loop system can be combined with high air pressure to produce gradients greater than 80. The drawbacks to this system are the possible introduction of air into the system, possible sample disturbance, and anisotropic consolidation of the specimens under the influence of the gradient. By applying high air pressure to the column of permeant, air

can dissolve into the permeant as shown by Henry's Law (Nebergall et al, 1976). Henry's Law states the amount of air dissolved into a fluid is proportional to the air pressure, contact time, and contact area. This air may enter the specimen and cause unsaturated conditions. Although this effect can be reduced by frequent changing of the fluid in the inflow standpipe, air may still infiltrate the specimen. The effects of an unsaturated soil on hydraulic conductivity are discussed in this paper in the section titled "Degree of Saturation". High gradients may also cause specimen disturbance or consolidation, as discussed in the section titled "Gradient".

CHAPTER VI

METHODOLOGY

Testing procedures on the Triaxial Permeability Device generally involve the following steps: sample mounting, application of cell pressure and backpressure, backpressure saturation, consolidation, and hydraulic conductivity testing. To aid in the description of these procedures, figure 6.1 shows numerical designations for each valve on the testing apparatus.

Specimen Mounting

The soil specimens are prepared, aged, and trimmed to size (2 inch diameter). Procedures for specimen preparation and aging will vary depending upon the scope of the analyses. Consideration should be given to the variables affecting conductivity, as discussed in the previous chapters. Prior to mounting, porous stones and filter paper are boiled in deaired, demineralized water and allowed to cool in the water. This procedure eliminates most of the entrapped air in the stones and filter paper.

Once the stones have cooled, the bottom stone is installed. A slight flow of water is applied from the influent reservoir to the bottom platen to eliminate any air in the inflow line. The porous stone is now placed on the

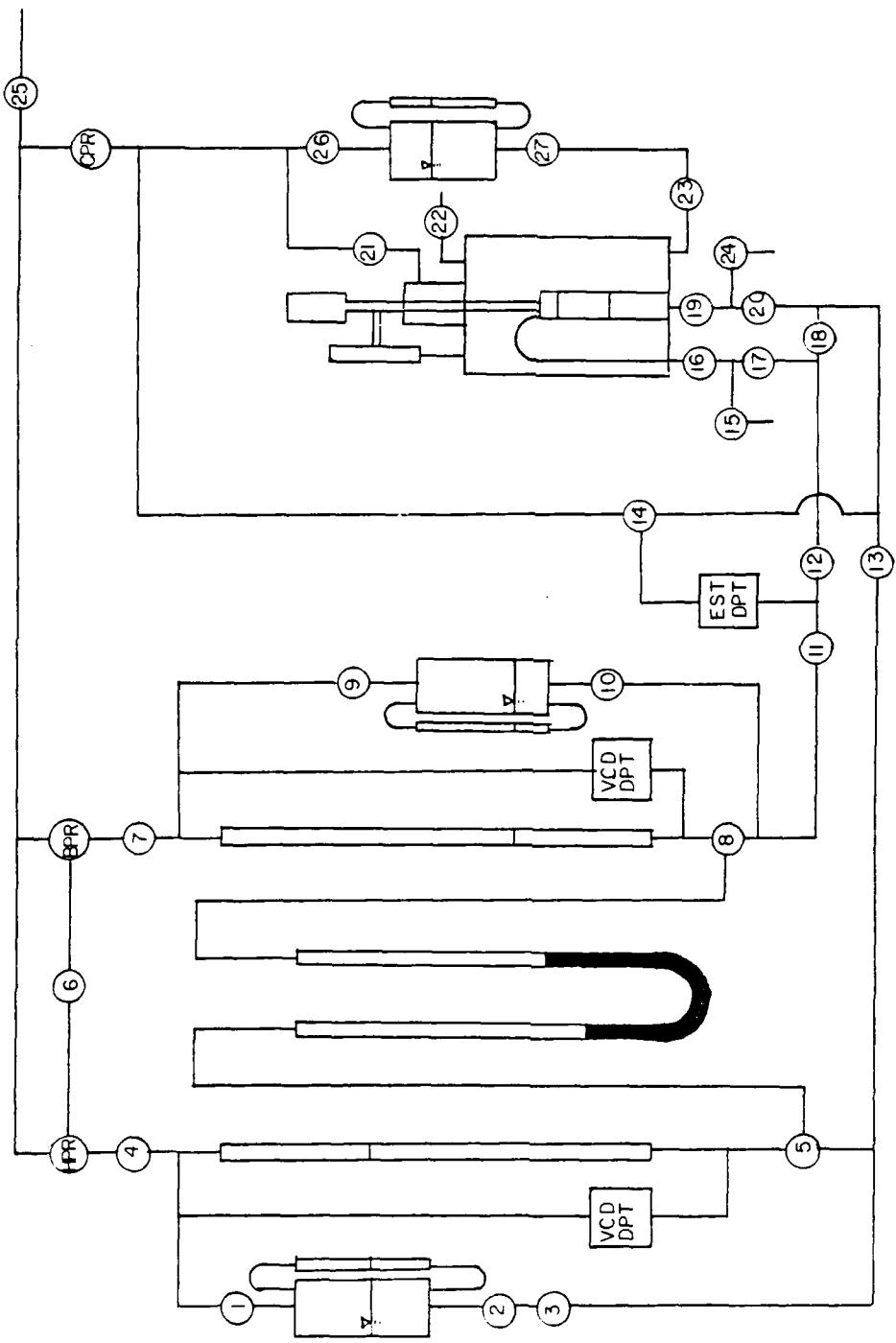


Fig. 6.1 Valve Numbering Scheme, Triaxial Permeability Device

bottom platen, and covered with filter paper. The inflow line is then closed. The soil specimen is placed on the bottom platen and another piece of filter paper is centered on top of the specimen, along with another porous stone. A slight fluid flow from the effluent line is used before the top platen is installed to eliminate air in the line.

A rubber (or other material) membrane is placed inside a 2.5 inch diameter tube and is turned over the ends. This tube is referred to as a membrane stretcher. Suction is applied to the space between the membrane and the tube. Suction may be applied from a vacuum line or by mouth. The expanded membrane is slid around the sample without touching it, and centered lengthwise over the sample. This procedure requires disconnection of the sample effluent line, at the point where the effluent line is connected to the triaxial cylinder bottom plate. The suction is then released so the membrane contracts around the specimen. If two membranes are desired, then the second membrane is installed over the first membrane. After the membrane stretcher is removed, two rubber (or other material) O-rings are stretched over the ends of the membrane stretcher. The tube is then placed around the specimen and the two O-rings are rolled off the tube, below the bottom porous stone. These O-rings should be over the membrane and as close to the bottom stone as possible. The tube is carefully lifted up from around the sample. The membrane is gently stroked from bottom to top

to release any air entrapped between the membrane and the specimen, and the two membranes. Two more O-rings are stretched around the membrane stretcher and installed above the top porous stone. The effluent line is then reattached.

The triaxial cell is assembled by placing the cylinder in the groove on the bottom plate, and the top plate is installed over the cylinder. The load rod is inserted through the top plate, making sure it is seated properly in the top platen. The top and bottom plates are then tightened together over the ends of the cylinder.

Once the sample is mounted and the triaxial cell is assembled, the cell is then filled with confining fluid. The inflow line to the top of the main cell recharge reservoir is removed so the reservoir can be filled with deaired, demineralized water. The plug valves (#23 and 27) below the reservoir are opened and water flows into the cell. The needle valve (#22) on top of the triaxial cell is opened to allow the displaced air to bleed off. Once the fluid level is within an inch of the top plate, flow to the triaxial cell is stopped by closing valve #27. All connections are then secured and the system is deaired. At this time backpressure saturation is initiated.

Backpressure Saturation

The saturation level of a soil specimen in a triaxial cell is commonly found by measuring the pore pressure response to a change in cell pressure. Specimens are saturated by maintaining a constant low effective stress on the soil while applying backpressure to the drainage lines (Lee et al, 1969). During this process data may be collected to calculate the Skempton pore pressure parameter B, which in turn gives an indication of the degree of saturation (Skempton, 1954).

Skempton derived an expression for the change in pore pressure (δU) of an undrained sample as:

$$\delta U = B(\delta \sigma_3 + A(\delta \sigma_1 - \delta \sigma_3)) \quad (6)$$

Since the soil is backpressure saturated under isotropic stress, $\delta \sigma_1 = \delta \sigma_3$, therefore:

$$\delta U = B(\delta \sigma_3) \quad (7)$$

or

$$B = \delta U / \delta \sigma_3 \quad (8)$$

In a saturated soil sample, any change in σ_3 should cause an equal change in the pore pressure, causing B to have a value of unity.

To start backpressure saturation valves #4, 7, 21, 25, and 26 are opened. The three way valves #1 and 9 are turned so air flow is to the top of the reservoirs. Three way

valve #8 is turned to allow flow to pass from the effluent standpipe to the specimen. Three way valve #14 is turned to allow flow from the cell pressure regulator to flow to the effective stress differential pressure transducer. Before cell pressure is applied or adjusted, the load rod is slightly seated into the top sample platen and fixed into position by tightening the set screw into the load rod. This prevents the cell pressure uplift from lifting the load rod out of position.

Valve #27 is opened and the cell pressure is adjusted to the desired level with the cell pressure regulator (σ_{cell}). Cell pressure is monitored with the cell pressure gage. Valve #11 is opened and the backpressure regulator is adjusted until the effective stress, as read from the effective stress transducer, is at the proper level. Valves #12, 16, 17, 18, 19, and 20 are opened at time $t=0$. This allows the stress level to be applied on the specimen. Cell pressure increases ($\delta\sigma_{cell}$) at constant effective stress are applied as follows: valves #12 and 23 are closed, the cell pressure is then increased to the desired level. The backpressure is adjusted until the desired effective stress is obtained. Set time $t=0$ and open valves #12 and 23 simultaneously. These procedures are repeated incrementally until a B value measurement is obtained.

Measurement of B Value

The Skempton pore pressure parameter B is defined in the section titled "Backpressure Saturation". This section outlines the procedures and calculations necessary to determine B value.

B value measurements can be made after the backpressure saturation procedures have begun. Although B values can be measured at any time during backpressure saturation, experience has shown that complete saturation of fine-grained soils is unlikely until the cell pressure is greater than or equal to about 40 psi. For coarse soils saturation may be achieved at lower cell pressures.

An arbitrary, low value of effective stress, σ_i , is maintained on the specimen during backpressure saturation. After the initial effective stress has been applied for at least ten minutes, valve #11 is closed for three minutes and a reading of cell pressure minus pore pressure (cp-u) is obtained from the effective stress transducer. At this time valve #12 is closed and valve #11 is opened. The effective stress transducer now reads cp-bp (bp=backpressure). Next, the cell pressure is increased by $\delta\sigma_3$ using the cell pressure regulator. At this time $cp-u = \delta\sigma_3 + \sigma_i$. Valve #11 is then shut and valve #12 is opened. The effective stress transducer now reads cp-u. This value is read after three minutes have elapsed. The change in effective stress between the two readings is $\delta\sigma$. The change in pore pressure

is calculated by:

$$\delta U = \delta \sigma_{cell} - \delta \sigma \quad (9)$$

The B value is then calculated using:

$$B = \delta U / \delta \sigma_{cell} \quad (10)$$

After the B value is measured, valve #12 is closed and valve #11 is opened. The backpressure is then increased until $cp - bp = \sigma_i$. This load is maintained on the specimen for at least ten minutes, and then the process is repeated. In fine grained soils it is advantageous to allow the pressure to remain of the sample for up to two hours before measuring B. Incremental pressure increases and B value measurements are continued until the desired B value is achieved. The final B value measurement should be taken at the stress level that will be used for mechanical consolidation and conductivity testing. Minimum B values of 0.95 and 0.92 are recommended for fine-grained and coarse grained soils, respectively.

Consolidation Analysis

After the desired level of saturation is obtained, the soil specimen may be isotropically or anisotropically consolidated to the desired level of stress. If the test results are used for design purposes, the stress level should equal the anticipated in-situ effective stress.

To start consolidation, valves #16 and 19 are closed (or #17 and 20) to isolate the specimen. Valves #11 and 12 are then opened. Three way valve #8 should be set so flow goes into the outflow standpipe. The load rod is then loaded to counteract the uplift forces in the cell. The weight placed on the load rod, p , is:

$$p = F(u) - W(app) \quad (11)$$

where: $W(app)$ = weight of load rod assembly, and
 $F(u)$ = cell pressure \times cross sectional area of
the load rod.

Another factor omitted from equation 11 is FRIC, which is equal to the load carried by the filter paper, membrane, and piston friction. Since filter paper is rarely used in the Triaxial Permeability Device, and linear ball bushings and teflon bushings limit load rod friction, the total of these effects is small in relation to the total load. Therefore, for the device described herein, the FRIC factor is negligible. Further details on these effects can be found in papers by Williams (1982) and Duncan and Seed (1967).

The load rod is carefully placed onto the top platen by releasing the pressure on the set screw. The dial gage or LDVT is set on the load rod and an initial reading is taken. Valve #10 is opened to lower the fluid level in the outflow

standpipe. Then valve #10 is closed and the initial height of fluid in the outflow standpipe is recorded followed by the initial value of the volume change using the differential pressure transducer.

At time $t=0$, valves #16 and 19 are opened (or #17 and 20) to allow two way drainage out of the sample. Readings of axial deformation and volume change are taken at the specified intervals so a plot of volumetric strain versus log time may be obtained. The axial and volumetric strain values are also used to evaluate the corrected cross-sectional area of the specimen, A_C (Lambe, 1951):

$$A_C = V_O(1-\epsilon_v)/(L_O(1-\epsilon_a)) \quad (12)$$

where:

V_O = initial sample volume

ϵ_v = volumetric strain

L_O = initial sample length, and

ϵ_a = axial strain.

Consolidation should continue until the rate of secondary compression can be calculated. When the consolidation is completed conductivity testing can begin.

Conductivity Testing

Conductivity testing requires the use of one of the three gradient loops, as introduced previously in the section titled "Gradient Control". After the level of

gradient to be used is selected, testing can begin on the appropriate loop.

Procedures for the use of each loop are outlined in this chapter, with the assumption that these procedures are for hydraulic conductivity testing. The test modifications required for chemical conductivity testing are also described in this chapter.

Low Gradient Open Loop Hydraulic Conductivity Test

For low gradient open loop testing, the specimen is isolated by closing valves #16, 18, and 19. Valves #11, 12 and 13 are then opened. The three way valves #5 and 8 are set so they direct flow to the influent and effluent standpipes. The fluid is drained from the effluent standpipe by opening valve #10. The level of fluid in that standpipe may be set by adjusting the height of the effluent reservoir. When this is complete, valve #10 is closed.

The gradient level is set as a result of the equation:

$$i = (h_1 - h_2)/L \quad (13)$$

where: h_1 = height of water in the influent standpipe

h_2 = height of water in the effluent standpipe

L = length of the sample

i = gradient.

In this equation, i , h_2 , and L are known and h_1 is the

variable. Valves #2 and 3 are opened allowing the water to flow into the influent standpipe until it reaches the desired height, h_1 . Valves #2 and 3 are then closed.

To solve for k , the following data are required: length of the specimen, cross sectional area of the specimen, initial height of the fluid in both standpipes, initial readings on the volume change transducers, the height of the fluid in each standpipe, and the volume change as a function of time.

At time $t=0$ for the permeability analysis, valves #16 and 19 are opened. The elapsed time and the height of fluid in each standpipe are recorded with time. The analysis is continued until approximately two pore volumes of water have passed through the specimen or until consistent values of k are achieved. When the fluid in the influent standpipe approaches zero, valves #16 and 19 are closed and the time and fluid levels are recorded. If testing is to continue, valves #2 and 3 are opened to refill the influent standpipe and valve #10 is opened to drain the effluent standpipe. The fluid levels in the standpipes are set as determined by equation 13. When this is complete, valves #2, 3, and 10 are closed and the fluid levels are recorded. Once again, valves #16 and 19 are opened simultaneously to initiate the permeability test. The process is continued until the required conductivity data are obtained. Conductivity is calculated as follows:

$$k = \frac{La_a a_b}{At(a_a + a_b)} \ln \frac{H_1}{H_2} \quad (14)$$

where: A = corrected cross-sectional area of the sample,

L = length of the sample,

t = time between readings,

$H_1 = h_1 - h_2$,

$H_2 = h_3 - h_4$,

a_a = cross sectional area of influent standpipe, and

a_b = cross sectional area of effluent standpipe.

Complete derivation of this formula is given in Appendix A.

Closed Loop Hydraulic Conductivity Test

For gradients less than 80, the closed loop system may be used. This procedure may begin after consolidation or after low gradient (<20) testing is complete.

Again, the sample is isolated by closing valves #16 and 19. Valves #11, 12, and 13 are then opened. The three way valves #5 and 8 should be set so flow is directed to the mercury manometer. The mercury heights are calculated from the gradient equation:

$$i = \delta H (G_{Hg} - 1) / L \quad (15)$$

where: δH = the difference in height of mercury in each leg of the manometer,

G_{Hg} = specific gravity of mercury, and

L = length of the specimen.

Valve #10 is opened. When the mercury height, δH is known, the gradient is set by carefully and slowly cracking valve #18. As fluid flows out, the height of the effluent side of the mercury column rises. Valve #18 is closed when the difference in height between the mercury columns reaches the desired level. The height of the mercury columns is recorded and at time $t=0$ valves #16 and 19 are opened. The time and height of the mercury columns are monitored. When the mercury columns approach the same height, valves #16 and 19 are closed and the time and column heights are recorded. The gradient may be reset by opening valve #10 and slowly opening valve #18. These valves should be shut when the height of the mercury reaches the proper level for the desired gradient. The analysis is continued until two pore volumes of fluid have passed through the specimen or until the conductivity is constant after a number of measurements. Conductivity is calculated by the following formula:

$$k = \frac{La_a a_b}{At(G_{Hg} - 1)(a_a + a_b)} \ln \frac{H_1}{H_2} \quad (16)$$

where: L = sample length,

A = corrected cross sectional area of the sample,

t = time between readings,

a_a = cross sectional area of influent mercury column,

a_b = cross sectional area of effluent mercury column,

$H_1 = (h_3 - h_1)(G_{Hg} - 1)$, and

$$H_2 = (h_4 - h_2)(G_{Hg} - 1).$$

Derivation of this formula is given in Appendix A.

High Gradient Open Loop Hydraulic Conductivity Test

The high gradient test may begin after consolidation or completion of lower gradient testing. The specimen should be isolated by closing valves #16 and 19. Three way valves #5 and 8 should be set to route flow to the standpipes. The desired gradient is used to calculate the required pressure as follows:

$$P \text{ (psi)} = i \tau_w L \quad (17)$$

where: i = gradient,

τ_w = unit weight of water,

L = length of sample, and

P = required pressure is psi.

The influent standpipe is filled and the effluent standpipe drained, as described in the section on low gradient testing, using valves #2, 3, and 10. The three way valve #14 should be adjusted so pressure comes from the influent line. Now the effective stress transducer reads the pressure difference (P) across the specimen. The high pressure regulator is adjusted until the effective stress transducer reads the pressure calculated for the desired gradient (from equation 17). The height of the water columns and the values from the volume change transducers

are then recorded. At time $t=0$, open valves #16 and 19. The time and water column levels are monitored periodically. As the water level in the inflow standpipe reaches zero, valves #16 and 19 are closed and the water column heights, volume change transducer readings, and time are recorded. The height of the water columns and the gradient are reset and the fluid levels and volumes are recorded. The analysis is continued until two pore volumes of water have passed through the specimen or until a constant k has been obtained. The hydraulic conductivity, k may be calculated by the following equation:

$$k = \frac{La}{At} \frac{a_a a_b}{a_a + a_b} \ln \frac{H_1}{H_2} \quad (18)$$

where: L = sample length,
 A = corrected cross sectional area of the sample,
 t = time between readings,
 a_a = cross sectional area of influent standpipe,
 a_b = cross sectional area of effluent standpipe,
 $H_1 = (h_1 - h_3) + (H_p/\tau - B_p/\tau)$, and
 $H_2 = (h_2 - h_4) + (H_p/\tau - B_p/\tau)$.

as derived in Appendix A.

Changing Permeant During Testing

Following a hydraulic conductivity test it may be desirable to perform a chemical conductivity analysis.

To perform a chemical conductivity analysis, the influent lines must be purged of water. To do this requires isolating the specimen by closing valves #16 and 19, and opening valves #2, 3, 13, 20, and 24. Three way valve #1 is opened so it vents the influent reservoir. Influent then discharges from the influent sampling port (#24). When the flow has ceased, close valve #24 and remove the fitting from the top of the influent reservoir. The influent reservoir is then filled with the new permeant. Open valve #24 and the air in the influent line discharges through the influent sampling port, followed by an outflow of the permeant. Some permeant should be discharged through the influent line to purge the air and water. Valve #24 is then closed. The influent reservoir is filled to the desired level with new permeant and the top fitting is reattached to the influent reservoir. Valve #1 is reset to direct air pressure to the influent reservoir.

Low Gradient Open Loop Conductivity Test With New Permeant

For the low gradient test, the specimen is isolated, the influent standpipe is drained, and the level of the effluent standpipe is lowered. The effluent standpipe is drained into the effluent reservoir and the water in the influent standpipe is discharged through valve #24. At this time the permeant in the influent reservoir is changed as described in the section titled "Changing Permeants During Testing". The residual water is flushed out of the influent

standpipe by filling and draining the standpipe three times with the new permeant. These washings are removed through valve #24. Conductivity testing may be resumed using the procedures specified for low gradient conductivity testing previously listed.

Closed Loop Conductivity Test With New Permeant

For a gradient <80, the specimen is isolated and the mercury level adjusted in the influent mercury manometer leg to the maximum height using air pressure from the backpressure regulator. This drives much of the water out of the influent leg of the manometer. The manometer is isolated by adjusting valves #5 and 8 to stop flow to the manometer. The influent lines and reservoir are then purged as previously outlined. The backpressure is reset to the proper effective stress level. Three way valves #5 and 8 are then opened and the conductivity test is resumed using the procedures outlined for the closed loop conductivity test.

High Gradient Open Loop Conductivity Test With New Permeant

For the high gradient test, the fluid is discharged from the inflow standpipe. The influent lines and reservoir are purged as previously outlined. The procedures follow those outlined for high gradient open loop conductivity testing.

Chemical Breakthrough

Even though the water has been purged from the influent lines to valve #24, the rest of the tubing going into and out of the sample is filled with water. Since flow proceeds at a slow rate, some time is required before the new influent reaches the soil specimen. Predicting the time for the permeant to reach the soil specimen requires calculation of the volume of water in the line from the tee between valves #19 and 20 to the top of the bottom porous stone. This value is estimated by measuring the length of tubing between these points. Dividing the length of tubing by the hydraulic conductivity of the soil yields the time required for the permeant to reach the soil sample.

Because of the effects the chemical permeant may have on the specimen, it is desirable to know the concentration of chemical in the pore fluid at any given time. When the pore fluid in the specimen is entirely composed of the chemical permeant, accurate values of chemical conductivity can be calculated. The condition of the pore fluid being 100% chemical permeant is termed 'breakthrough'. Evaluation of this condition is expressed in terms of the breakthrough curve.

The breakthrough curve is a plot of percent new permeant concentration in the pore fluid versus pore volumes of flow. By sampling the effluent through the effluent sampling port, values for the permeant concentration can be

obtained. Since the effluent sampling port is separated from the sample by a length of tubing, the concentration of permeant in the effluent cannot be determined until more than one pore volume of permeant has passed through the specimen.

When the new permeant is at the base of the specimen, summation of the pore volume of the specimen, volume of tubing between the top porous stone and the effluent sampling port, and volume of the effluent sample yields the volume of fluid that must pass through the sample before chemical concentration can be measured. Subsequent measurements are made once a pore volume or fraction of pore volume has passed through the sample, provided the size of the sample for chemical concentration analysis remains the same.

The size of the effluent sample has an effect on the chemical concentration. The accuracy of the analysis increases as the size of the effluent sample decreases. This effluent sample yields an approximate but reliable estimation of chemical concentration at the top of the sample at some known time.

Sampling Procedures

Before using the Triaxial Permeability Device, the volume of fluid required to fill the tubing from the center of the tee between valves #19 and 20 to the top of the bottom porous stone is measured. The fluid volume required

to fill the tubing from the tee between valve #16 and 17 to the effluent sampling port is also measured.

The chemical conductivity analysis is begun as specified previously in the section titled "Changing Permeants During Testing". The new permeant reaches the specimen when the change in effluent volume is equal to the volume of fluid in the tubing from the tee between valves #19 and 20 to the base of the specimen. When the new permeant reaches the tee between valves #16 and 17, the effluent may be sampled for chemical concentration.

The effluent may be measured at any time, however valves #16 and 19 must be closed and the height of fluid in the standpipes and time must be recorded. The fluid volume trapped in the tee between valves #16 and 17 and the effluent sampling port must be drained off. At this time the desired effluent sample is at the mouth of the effluent sampling port. The desired volume of effluent is obtained by slowly opening valve #15. Once this is complete, conductivity testing may be resumed.

CHAPTER VII

LABORATORY RESULTS

KaoliniteHydraulic Conductivity

Initial testing with the Triaxial Permeability Device was performed on a kaolinite specimen obtained in Southern Georgia. This core was taken from a depth of approximately 40 feet. The sample was backpressure saturated and isotropically consolidated at an effective stress of 35 psi. Hydraulic conductivity tests were then performed using the closed loop system at gradients of 45 and 24.

When a gradient of 45 was used, the hydraulic conductivity was 7×10^{-8} cm/sec. At a gradient of 24, the hydraulic conductivity was 6×10^{-8} cm/sec. The difference in conductivities between the two tests is likely due to variations in the soil structure, density, and void ratio. This small difference in values of this magnitude is assumed to be negligible.

Chemical Conductivity

A chemical conductivity test was performed on a different kaolinite specimen from the same core used in the hydraulic conductivity test. This specimen was also backpressure saturated and isotropically consolidated at an effective stress of 35 psi. Testing was performed at a

gradient of 200. The permeant used in this test was a solution of sulfuric acid mixed with deaired, demineralized water, and the pH of the solution was 2.0.

A total of 24 permeability tests were performed over a three week period using the high gradient, open loop system. The average chemical conductivity for this test was 7.5×10^{-8} cm/sec. The chemical conductivity was relatively constant over the entire testing period. A total of 1.6 pore volumes of chemical solution were permeated through the sample. The chemical conductivity was about equal to the hydraulic conductivity, and the difference may be attributed to the fact that different specimens were used for the hydraulic and chemical conductivity analyses.

Bentonite

Hydraulic Conductivity

Hydraulic conductivity tests were also performed on bentonite specimens prepared from a slurry. These specimens were prepared by placing hydrated bentonite in a plexiglass cylinder and consolidating under an axial load. Filter paper was placed around the sample to shorten the drainage paths, and porous stones were placed on each end of the specimen. These tests were difficult to perform, mostly due to the nature of the clay.

In the first test, the bentonite specimen was placed in the triaxial cell before it had reached secondary

consolidation. The specimen was backpressure saturated and then the consolidation analysis was initiated. Without the benefit of filter paper along the sides to speed drainage, and because of the low hydraulic conductivity of the bentonite, the primary consolidation continued for weeks. The diameter of the specimen began decreasing at the center of the specimen. Since the time to reach secondary compression could not be predicted, and because of the changes in the specimen shape, the test was stopped.

A second problem in testing the bentonite was bacteriological growth within the sample. In the second test, the specimen initially had a hydraulic conductivity in the order of $10E-9$ cm/sec. As the test progressed, the hydraulic conductivity decreased to $10E-10$ cm/sec. The test was stopped, and examination of the specimen for the pore fluid to be dark in color and have the characteristic odor of organic material. This bacteriological growth caused clogging of the pores and reduced the hydraulic conductivity of the soil. Therefore, in future tests, additives must be placed in the influent fluid to prevent bacteriological growth.

The growth of microorganisms during testing may clog the flow channels in a soil (Olson and Daniel, 1981). Allison (1947) found phenol at concentrations of 1000 ppm and formaldehyde at concentrations of 2000 ppm were effective in reducing the growth of microorganisms. In

future testing with the Triaxial Permeability Device the influent water will be treated with 0.5% sodium hypochlorite.

Discussion

Hydraulic Conductivity Test on Kaolinite

Foreman and Daniel, 1986, performed a hydraulic conductivity test on kaolinite using a flexible walled permeameter. Their sample was prepared from a pulverized soil in a Proctor mold using standard (ASTM D698) compactive effort. This test was performed at a variety of gradients with an effective stress of 15 psi. Their results show a hydraulic conductivity of 3.5×10^{-7} cm/sec for gradients of 24 and 45. The results for the hydraulic conductivity of kaolinite tested in the Triaxial Permeability Device was 7×10^{-8} cm/sec. This specimen was tested at an effective stress of 35 psi. This follows the statement by Zimmie (1981) that hydraulic conductivity typically decreases with increasing effective stress.

Chemical Conductivity Test of Kaolinite

The results of these analyses are consistent with previous research. Murry (1951) found that kaolinite resists prolonged exposure to sulfuric acid. Carroll and Starkey (1971) also tested kaolinite by mixing the soil with solutions having a pH range from 1 to 12. Their results show low pH solutions have little effect on kaolinite.

Therefore, the acid used in our test had little effect on the composition of the kaolinite particles.

Although the composition of the kaolinite particles is not significantly affected by the acid, the affects of the acid on the clay structure must also be examined. Sridharan and Venkatappa Rao (1973) tested for changes in clays as a function of the dielectric constant of the permeant. They reported the structure of kaolinite is governed by the shear strength of the interparticle bonds. Sridharan et al found that as the dielectric constant of the permeant decreased, the shear strength of the interparticle bonds increased. Since dilute sulfuric acid has a dielectric constant equal to or slightly less than that of water, the permeant used in this test would be expected to have little effect on the interparticle bonds or the double layer thickness. Therefore, the dilute sulfuric acid would not cause a change in the structure of the kaolinite, and preferential flow paths would not be formed.

The gradient may also have an effect on the chemical conductivity of clay. In order to reduce testing time, a large gradient (200) was used. One problem with using large gradients is the potential for particle migration which can lead to piping, clogging, or hydraulic fracturing. Since the chemical conductivity under the high gradient is approximately equal to the hydraulic conductivity at low gradient, and the sample did not undergo gradient induced

consolidation or expansion, it may be concluded that this gradient did not cause piping, clogging, or hydraulic fracturing for this particular analysis.

CHAPTER VIII

RECOMMENDATIONS FOR FURTHER STUDY

Equipment Modifications

With the development and initial use of this testing device, certain modifications may be necessary to improve and simplify the operation of the device.

First, the load rod is subjected to mechanical wear through contact with the ball bushings in the load rod guide. To reduce wear and grooving by the ball bushings, the load rod must be case hardened. Because of this, type 344 stainless steel was used for the load rod since it may be more effectively case hardened than type 304 stainless steel. This creates a problem because type 344 stainless steel corrodes more readily than type 304 stainless steel, and the load rod is continuously immersed in the confining fluid. Therefore, the part of the load rod that is immersed in the confining fluid needs to be coated to limit the corrosion. The upper part of the load rod cannot be coated because it would interfere with the operation of the ball bushings. Another option would be to add a corrosion inhibitor to the confining fluid, providing it does not contribute to the degradation of the specimen membrane.

With the current configuration, the changing of permeants during testing introduces the new permeant up to the influent sampling port. This leaves water in the influent line from the influent sampling port to the bottom of the porous stone. In low conductivity soils, it may take a relatively long period of time for the new permeant to reach the soil specimen. To eliminate this delay, another line should be installed from the bottom sample platen to the outside of the triaxial cell. This way, permeant could be flushed through the influent line, through the bottom porous stone, and out the new line. This places the new permeant at the base of the specimen, reducing the time required for chemical conductivity testing.

Another problem identified with this testing device is that the weight of the top sample platen may put a significant stress on a soft or sensitive soil specimen. To prevent this, the end of the load rod may be threaded into the top sample platen. The top platen may then be supported by the load rod and not the specimen.

Finally, to prevent the problem of bacteriological growth in the specimen, the pore fluid may be conditioned, as discussed in the previous chapter. This is required for hydraulic conductivity testing, and may or may not be required for chemical conductivity testing, depending upon the toxicity of the chemical.

Additional Testing

To demonstrate the effectiveness of the Triaxial Permeability Device, and to provide useful hydraulic and chemical conductivity data, further testing with this device is necessary. This testing may be performed on soils typically used in clay liners, and permeants that are relatively abundant in waste fills.

The additional testing may be performed using illite, kaolinite, attapulgite, or smectite clays. The permeants used for chemical conductivity testing may include aromatic hydrocarbons (e.g. xylene), ketones (e.g. acetone), and alcohols (e.g. ethylene glycol). Testing may be done with variations in soil stresses, chemical concentrations, and gradient.

CHAPTER IX

CONCLUSIONS

A state-of-the-art method for determining the hydraulic and chemical conductivity of fine-grained soils has been presented in this paper. The factors that primarily affect hydraulic and conductivity were presented, along with current methods of conductivity testing. The design of the Triaxial Permeability Device was introduced, and each major component of the device was discussed. Also, methodology for use of the Triaxial Permeability Device was given, along with limited laboratory results and recommendations for continued study.

The primary variables affecting the hydraulic conductivity of fine-grained soils are void ratio, soil structure, state of stress, degree of saturation, thixotropy, and gradient. Current test methods include using fixed or flexible walled permeameters. Use of a fixed walled permeameter requires trimming the specimen to the exact diameter of the permeameter, or compacting the soil directly into the permeameter. Specimens trimmed to a desired diameter and placed in a fixed walled cell may not have intimate contact with the cell walls. When soils are compacted in the cell, it is very difficult to control the void ratio and soil structure. To eliminate these problems,

a flexible walled permeameter should be used. The specimen may be trimmed to the required diameter, allowing use of a soil of desired void ratio and soil structure, and providing intimate contact between the cell walls and the specimen. Since the soil is not compacted into the cell, the void ratio and soil structure remain intact.

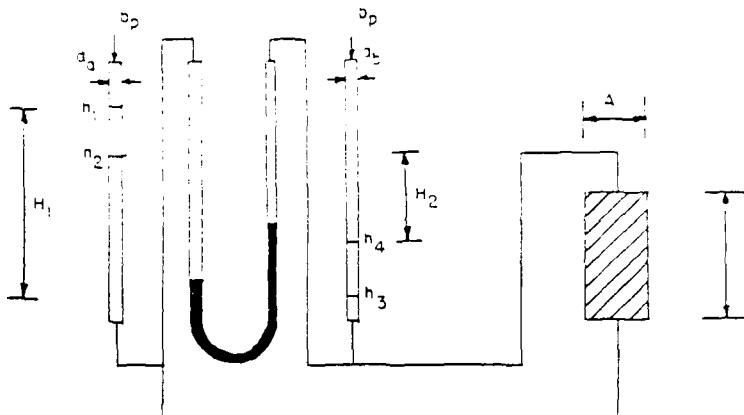
The state of stress of a soil specimen also affects conductivity. The triaxial cell is the only permeameter that provides complete control of axial, lateral, and effective stresses. Also, the degree of saturation can be easily measured in a triaxial cell, and a desired level of saturation can be achieved in a triaxial cell through backpressure saturation. These factors lead to the use of the triaxial cell as the permeameter in this device.

Three different methods may be used to impose gradients on the soil specimens, depending primarily on the desired magnitude of the gradient. The closed loop system may be used to eliminate contact of the permeant with air. Also, the use of electronic devices to monitor the stress levels, gradient, and volume change allow for accurate and continuous monitoring of pertinent test data.

The system is designed such that new permeants may be introduced without changing the soil stresses, degree of saturation, gradient, or other test parameters.

This research has evaluated the factors affecting hydraulic and chemical conductivity, and developed a testing device and methodology to perform conductivity tests providing maximum control over these factors. Based on the research performed for this study, the Triaxial Permeability Device may be used successfully for the evaluation of the conductivity of fine-grained soils while allowing variations in stresses, gradient, and permeants.

APPENDIX A
FORMULA DERIVATIONS



OPEN SYSTEM LOW GRADIENT

INITIAL CONDITIONS

$$H_{inf_i} = b_p/\gamma + h_1 \quad H_{eff_i} = b_p/\gamma + h_3$$

$$H_1 = H_{inf_i} - H_{eff_i} = b_p/\gamma + h_1 - b_p/\gamma - h_3 = h_1 - h_3$$

FINAL CONDITIONS

$$H_{inf_f} = b_p/\gamma + h_2 \quad H_{eff_f} = b_p/\gamma + h_4$$

$$H_2 = H_{inf_f} - H_{eff_f} = b_p/\gamma + h_2 - b_p/\gamma - h_4 = h_2 - h_4$$

DARCY'S LAW

$$Q = -kiAt \quad i = H/L$$

$$dQ = -kiAdt = adj = a_a(h_1 - h_2) = a_b(h_4 - h_3)$$

$$h_4 - h_3 = a_a/a_b (h_1 - h_2)$$

$$H_1 - H_2 = h_1 - h_3 - h_2 + h_4$$

$$= h_1 - h_2 + a_a/a_b(h_1 - h_2)$$

$$= (h_1 - h_2)(1 + a_a/a_b)$$

$$dQ = a_a(h_1 - h_2)$$

$$= a_a(H_1 - H_2)/(1 + a_a/a_b) = -kHAdt/L$$

$$a_a dH/(1 + a_a/a_b) = -kHAdt/L$$

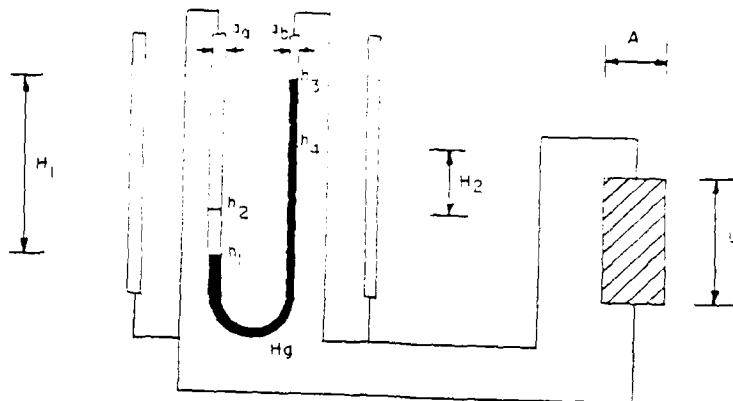
$$\int_{H_1}^{H_2} \frac{dH}{H} = - \int_0^t (1/a_a L) kA(1 + a_a/a_b) dt$$

$$\ln H_1/H_2 = (1/La_a) kAt (1 + a_a/a_b)$$

$$k = (La_a/(At(1 + a_a/a_b))) \ln H_1/H_2$$

$$\text{where: } H_1 = h_1 - h_3$$

$$H_2 = h_2 - h_4$$



CLOSED SYSTEM

DARCY'S LAW $Q = -kiAt$ $dQ = -kiAdt$

ASSUME: Volume of Inflow = Volume of Outflow

$a_a(h_2 - h_1) = a_b(h_3 - h_4)$

$h_2 - h_1 = a_b/a_a (h_3 - h_4)$

$$\begin{aligned}
 \text{Gradient} = & \quad H_1 = (h_3 - h_1)(G_{Hg} - 1) \\
 & \quad H_2 = (h_4 - h_2)(G_{Hg} - 1) \\
 H_1 - H_2 = & (h_3 - h_1)(G_{Hg} - 1) - (h_4 - h_2)(G_{Hg} - 1) \\
 & = (G_{Hg} - 1)((h_3 - h_4) + (h_2 - h_1)) \\
 & = (G_{Hg} - 1)((h_3 - h_4) + a_b/a_a(h_3 - h_4)) \\
 & = (G_{Hg} - 1)((h_3 - h_4)(1 + a_b/a_a))
 \end{aligned}$$

$dQ = a_b(h_3 - h_4) = -kHA(1/L)dt$

$h_3 - h_4 = (H_1 - H_2)/((G_{Hg} - 1)(1 + a_b/a_a))$

$dH = H_1 - H_2$

$-a_b(H_1 - H_2)/((G_{Hg} - 1)(1 + a_b/a_a)) = kHA(1/L)dt$

$-a_bL/(A(G_{Hg} - 1)(1 + a_b/a_a)) \int_{H_2}^{H_1} dH/H = \int_0^t kdt$

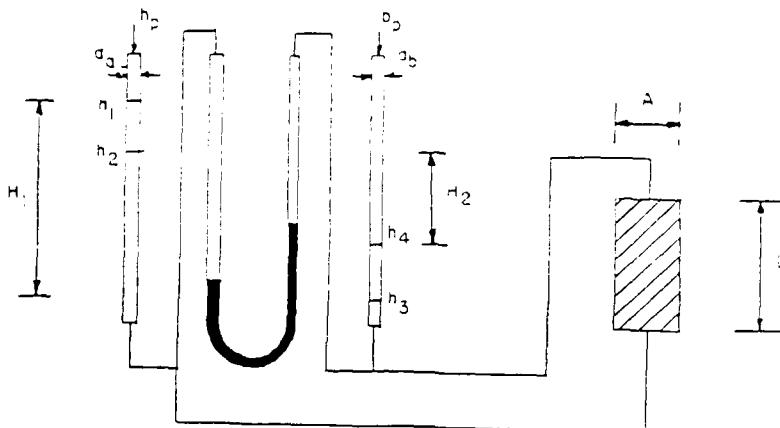
$(a_bL/(A(G_{Hg} - 1)(1 + a_b/a_a))) \ln H_1/H_2 = kt$

$k = (a_bL/(At(G_{Hg} - 1)(1 + a_b/a_a))) \ln H_1/H_2$

or $k = (La_a a_b / (At(G_{Hg} - 1)(a_a + a_b))) \ln H_1/H_2$

where: $H_1 = (h_3 - h_1)(G_{Hg} - 1)$

$H_2 = (h_4 - h_2)(G_{Hg} - 1)$



OPEN SYSTEM HIGH GRADIENT

INITIAL CONDITIONS:

$$\begin{aligned}
 h_{\text{inf},i} &= h_p/\gamma_{\text{inf}} + h_1 & h_{\text{eff},i} &= b_p/\gamma_{\text{eff}} + h_3 \\
 h_1 &= h_{\text{inf},i} - h_{\text{eff},i} = h_p/\gamma_{\text{inf}} + h_1 - b_p/\gamma_{\text{eff}} - h_3 \\
 &= h_1 - h_3 + h_p/\gamma_{\text{inf}} - b_p/\gamma_{\text{eff}}
 \end{aligned}$$

FINAL CONDITIONS:

$$\begin{aligned}
 h_{\text{inf},f} &= h_p/\gamma_{\text{inf}} + h_2 & h_{\text{eff},f} &= b_p/\gamma_{\text{eff}} + h_4 \\
 h_2 &= h_{\text{inf},f} - h_{\text{eff},f} = h_p/\gamma_{\text{inf}} + h_2 - b_p/\gamma_{\text{eff}} - h_4
 \end{aligned}$$

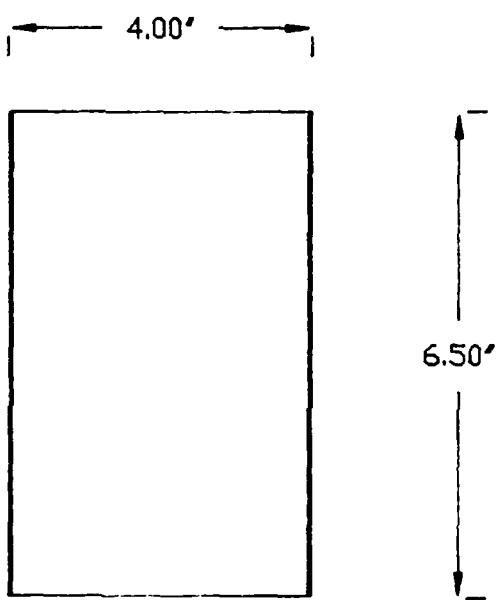
DARCY'S LAW

$$Q = -kiAt \quad dQ = -kiAdt \quad i = H/L$$

Assume: Volume of Inflow = Volume Outflow

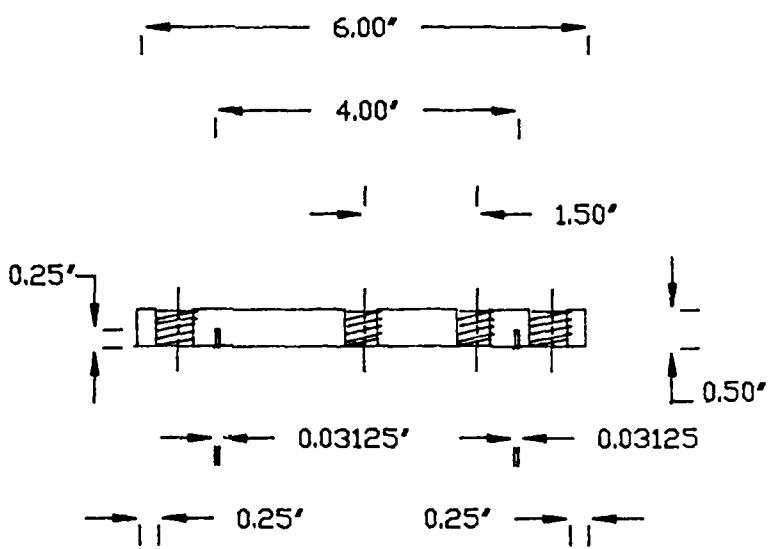
$$\begin{aligned}
 \text{In Standpipe} \quad dQ &= adh = a_a(h_1 - h_2) = a_b(h_4 - h_3) \\
 h_4 - h_3 &= (a_a/a_b)(h_1 - h_2) \\
 h_1 - h_2 &= h_1 - h_3 - h_2 + h_4 + (h_p/\gamma_{\text{inf}} - b_p/\gamma_{\text{eff}}) - (h_p/\gamma_{\text{inf}} - b_p/\gamma_{\text{eff}}) \\
 h_1 - h_2 &= h_1 - h_2 + (a_a/a_b)(h_1 - h_2) \\
 dQ &= a_a(h_1 - h_2) \\
 dQ &= a_a(H_1 - H_2)/l + a_a \cdot a_b = -kHA(1/l)dt \\
 (H_1 - H_2)/H &= -kA(1 + a_a/a_b)(1/(La_a))dt \\
 \int_{H_2}^{H_1} \frac{dH}{H} &= - \int_0^t \frac{1}{(La_a)} kA(1 + a_a/a_b) dt \\
 \ln \frac{H_1}{H_2} &= kAt/(1 + a_a/a_b)/(1/la_a) \\
 k &= (l/At)(1 + a_a/a_b)la_a \ln \frac{H_1}{H_2} \\
 \text{or} \quad k &= l/At(a_1 + a_2/a_3) \ln \frac{H_1}{H_2} \\
 \text{where:} \quad h_p &= h_1 - h_2 + h_4 - h_3 = b_p/\gamma_{\text{eff}} \\
 h &= h_1 - h_2 + h_4 - h_3 = b_p/\gamma_{\text{eff}}
 \end{aligned}$$

APPENDIX B
MACHINE DRAWINGS, TRIAXIAL CELL



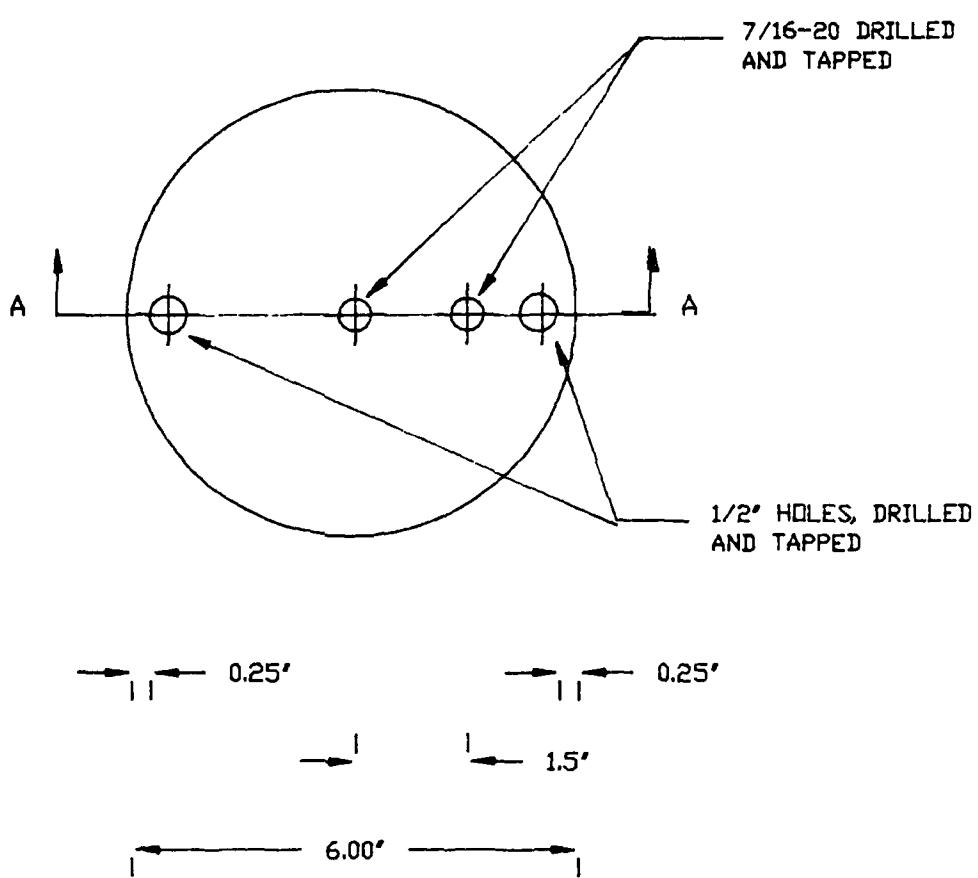
SECTION A - A

RESERVOIR CELL

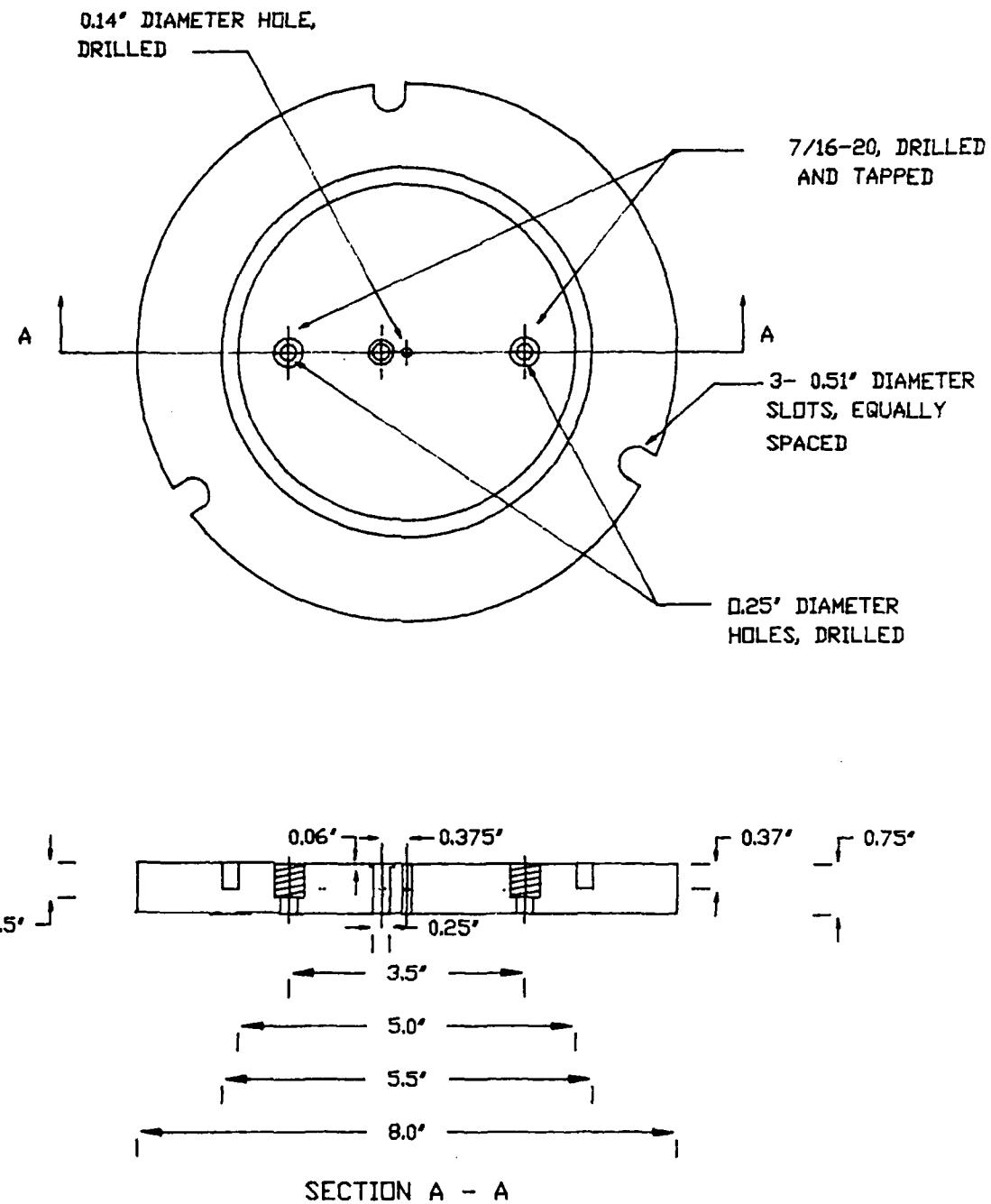


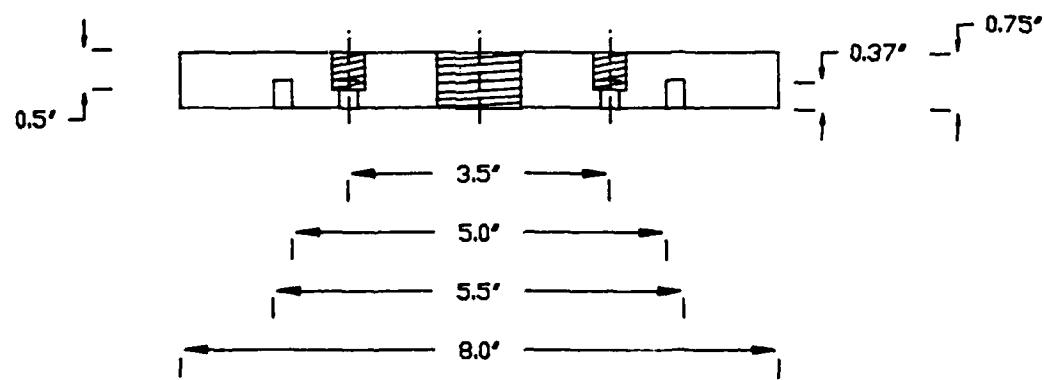
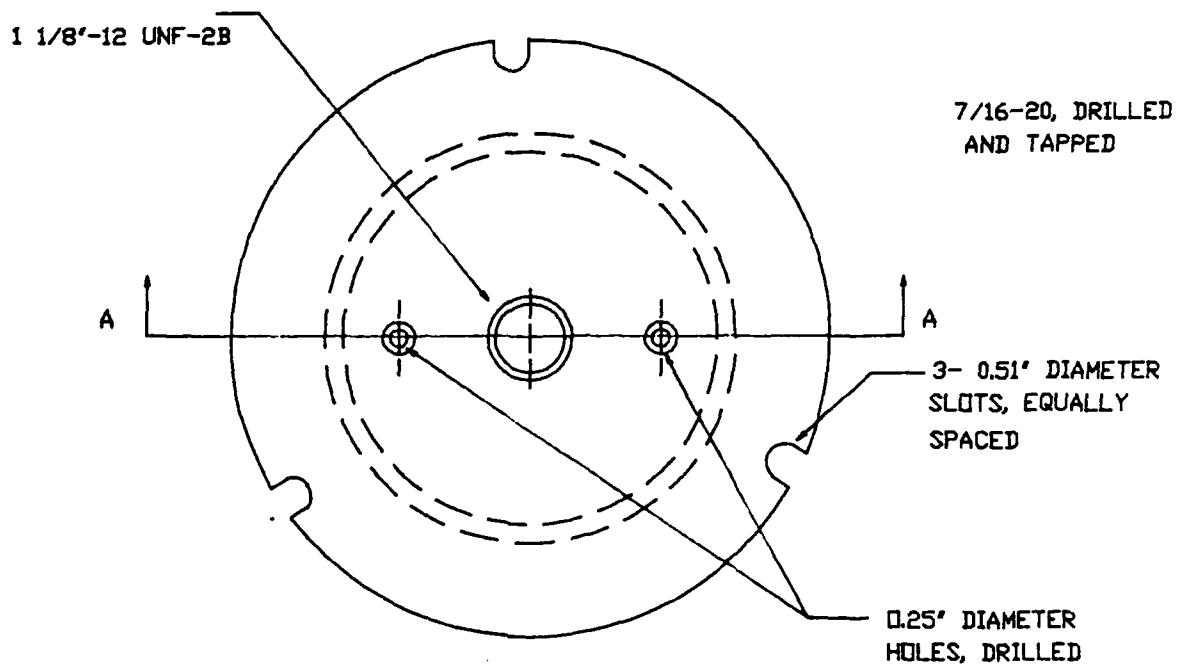
SECTION A - A

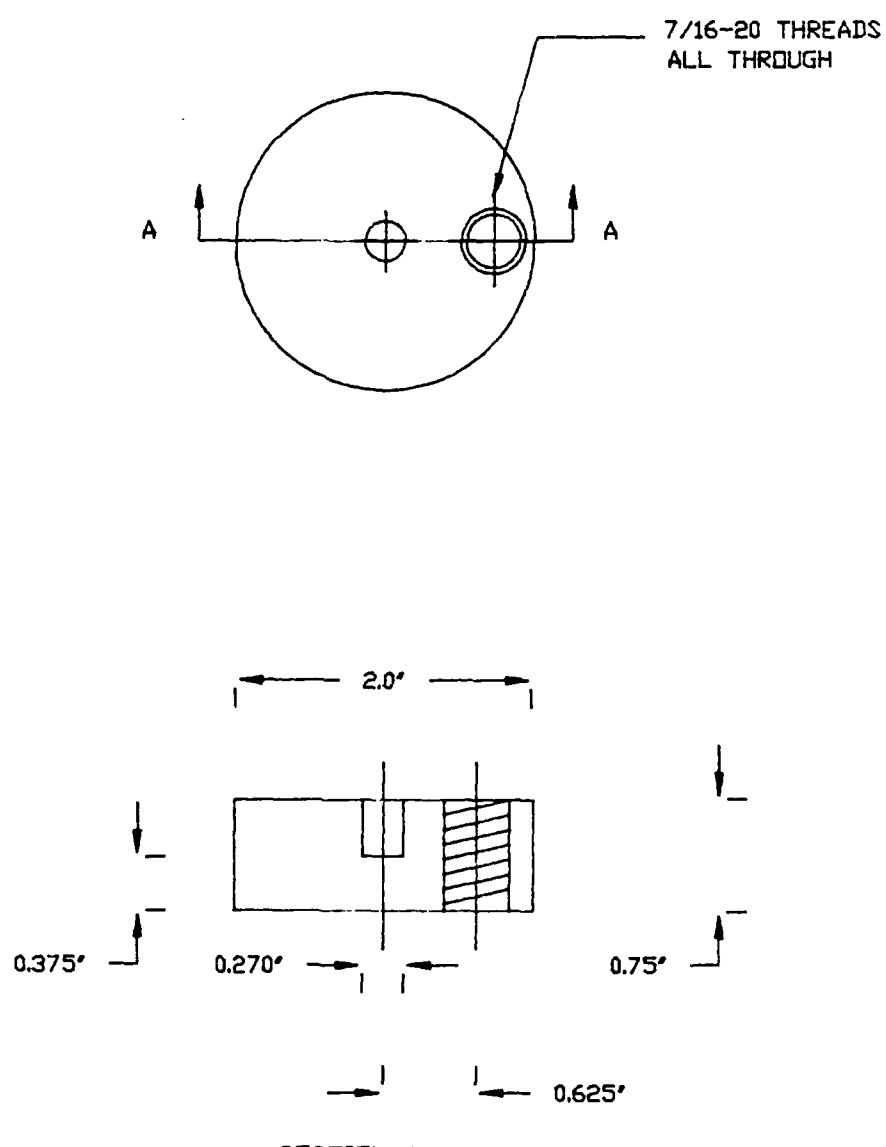
RESERVOIR TOP/BOTTOM PLATE



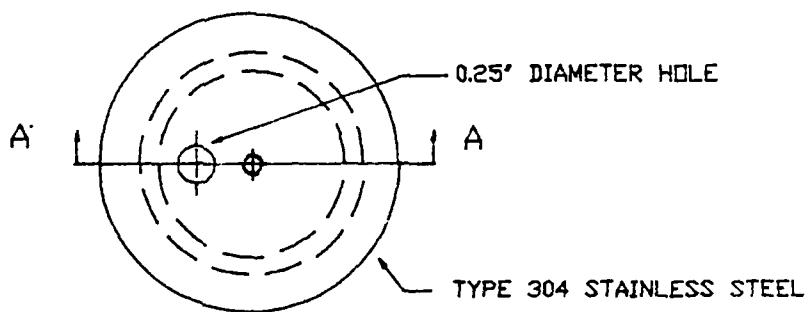
RESERVOIR TOP/BOTTOM PLATE



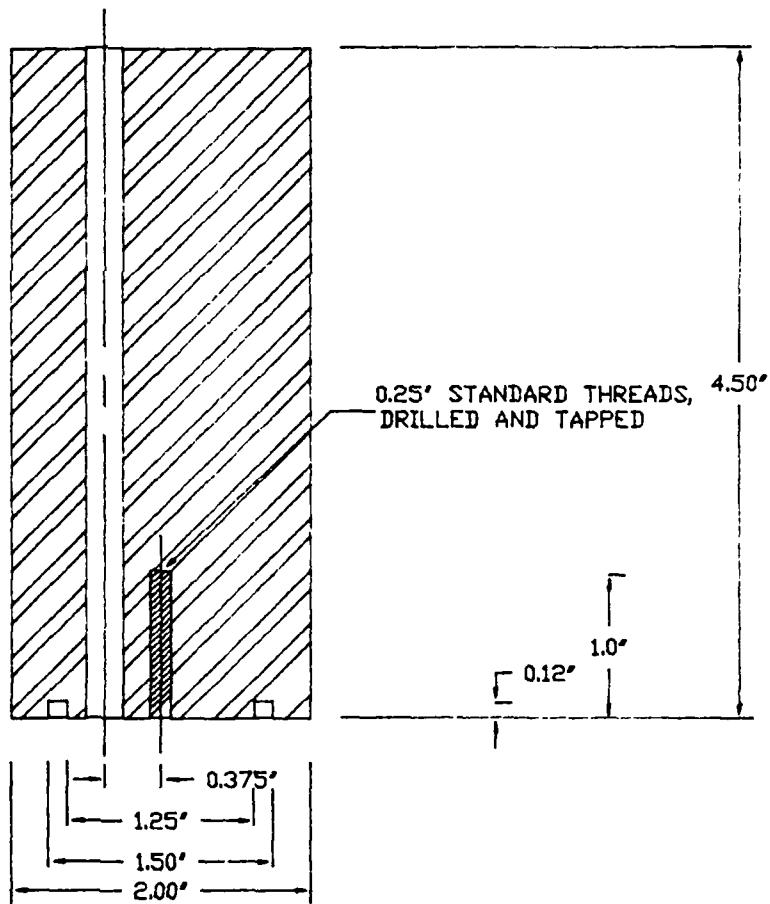




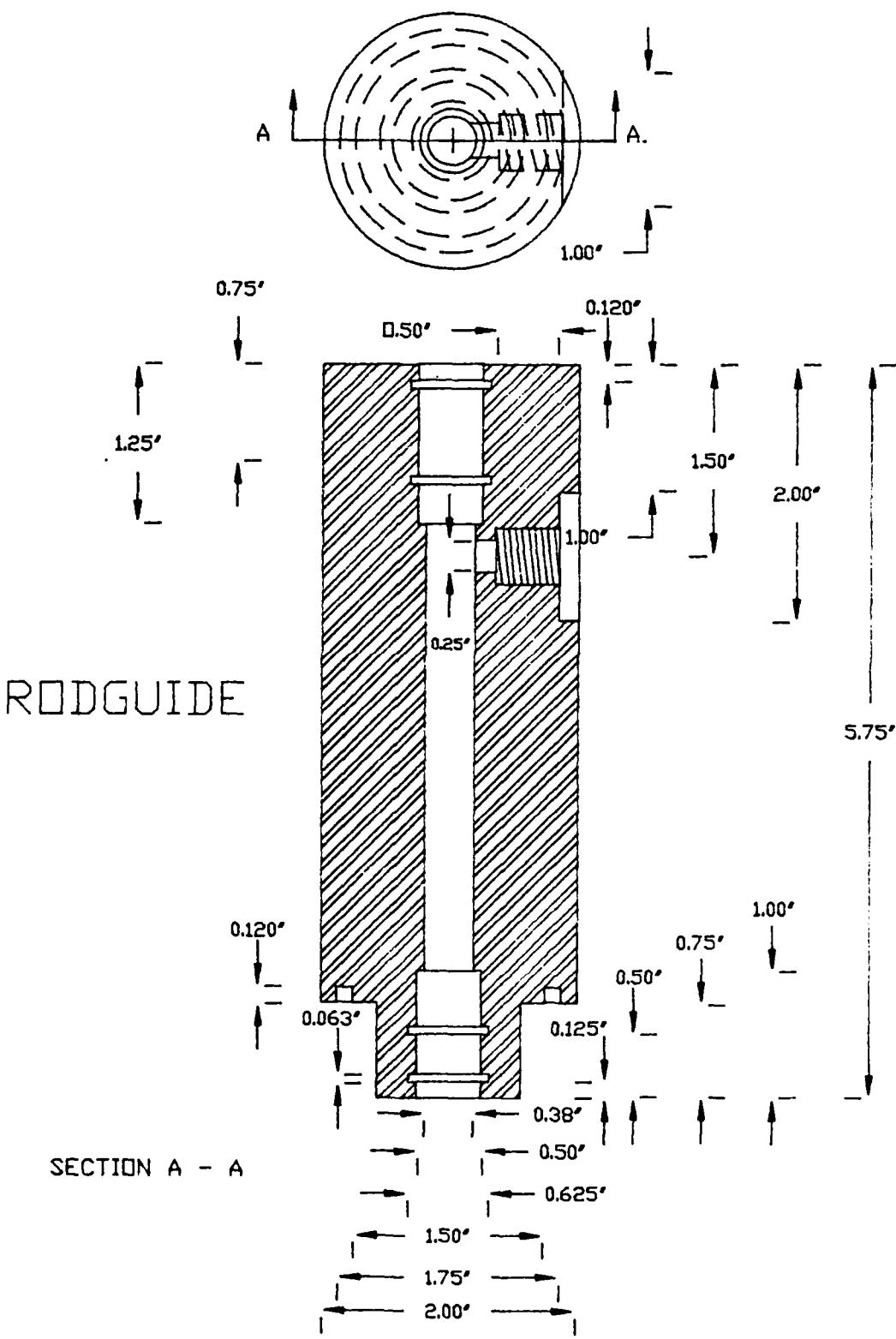
TOP SAMPLE PLATEN

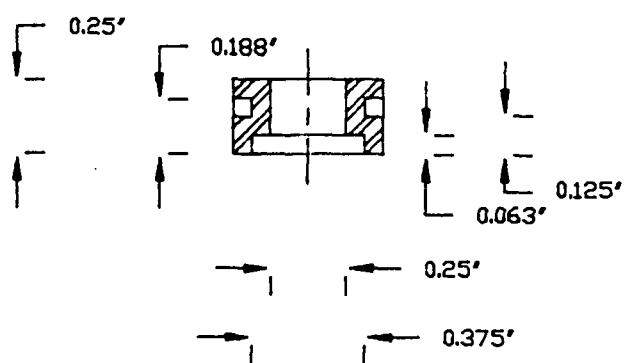
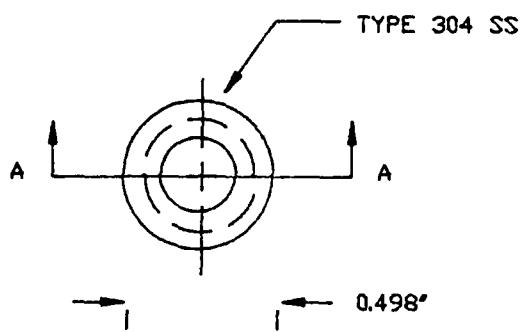


BOTTOM
PLATEN



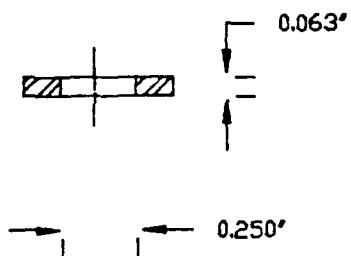
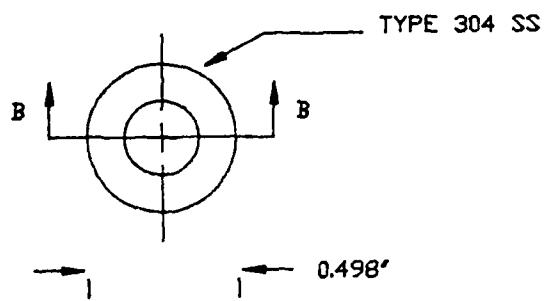
SECTION A - A





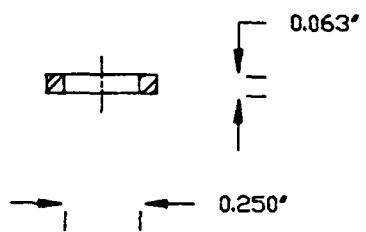
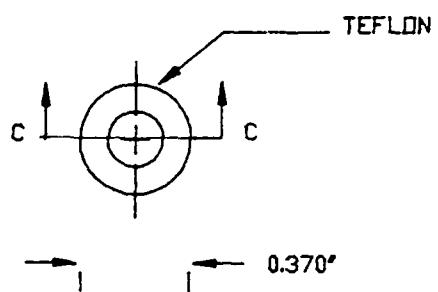
SECTION A - A

ROD GUIDE COLLAR



SECTION B - B

ROD GUIDE COLLAR



SECTION C - C

ROD GUIDE COLLAR

BIBLIOGRAPHY

1. Allison, L. E., 1947, Effect of Microorganisms on Permeability of Soil Under Prolonged Submergence", *Soil Science*, Vol. 63, pp. 439-450.
2. American Society for Testing and Materials, 1979, "Annual Book of ASTM Standards, Part 19", Philadelphia, PA.
3. American Society for Testing and Materials, 1970, "Special Procedures for Testing Soil and Rock for Engineering Purposes", ASTM STP #479, 5th ed.
4. Anderson, D., and Brown, K. W., 1981, "Organic Leachate Effects on the Permeability of Clay Liners", *Land Disposal: Hazardous Wastes, Proceedings of the Seventh Annual Research Symposium*, EPA-600/9-81-002b, pp. 119-130.
5. Anderson, D., Brown, K. W., and Green, J., 1982, "Effect of Organic Fluids on the Permeability of Clay Soil Liners", *Land Disposal of Hazardous Wastes, Proceedings of the Eighth Annual Research Symposium*, EPA-600/9-82-002, pp. 179-190.
6. Barden, L., and Sides, G. R., 1970, "Engineering Behavior and Structure of Compacted Clay", *Journal of the Soil Mechanics and Foundations Engineering Division, ASCE*, SM4.
7. Bishop, A. W., and Henkel, D. J., 1962, "The Measurement of Soil Properties in the Triaxial Test", 2 nd ed., Edward Arnold (publishers), Ltd.
8. Burmister, D. M., 1954, "Principles of Permeability Testing of Soils", *Symposium on Permeability of Soils*, ASTM STP #163, American Society of Testing and Materials, Philadelphia, PA., pp. 3-20.
9. Carroll, D., and Starkey, H. C., 1971, "Reactivity of Clay Minerals with Acids and Alkalies", *Clays and Clay Minerals*, Vol. 19, pp. 321-333.

10. Christiansen, H. R., 1944, "Permeability-Capillary Potential Curves for Three Prairie Soils", *Soil Science*, Vol. 57, pp. 321-333.
11. Darcy, H., 1856, "Les Fountaines Publique de la Dijon (The Water Supply of Dijon)", Dalmont, Paris.
12. Duncan, J. M., and Seed, H. B., 1967, "Corrections for Strength Test Data", *Proceedings, ASCE, Journal of Soil Mechanics and Foundations Division*, Vol. 93, No. SM5.
13. Dunn, R. J., 1983, "Hydraulic Conductivity of Soils in Relation to Subsurface Movement of Hazardous Wastes", Ph.D. Thesis, University of California, Berkley.
14. Dunn, R. J., and Mitchell, J. K., 1984, "Fluid Conductivity Testing of Fine-Grained Soils", *Journal of Geotechnical Engineering, ASCE*, Vol. 110, No. 11.
15. Foreman, D. E., and Daniel, D. E., 1986, "Permeation of Compacted Clay with Organic Chemicals", *Journal of Geotechnical Engineering, ASCE*, Vol. 112, No. 7, pp. 669-681.
16. Green, W. J., Lee, G. F., and Jones, R. A., 1979, "Impact of Organic Solvents on the Integrity of Clay Liners for Industrial Waste Disposal Pits: Implications for Groundwater Contamination", EPA Report PB81-213423.
17. Green, W. J., Lee, G. F., and Jones, R. A., 1981, "Clay-Soils Permeability and Hazardous Waste Storage", *Journal, Water Pollution Control Fed.*, 53:1347-54.
18. Lambe, T. W., 1951, "Soil Testing For Engineers", John Wiley and Sons, Inc.
19. Lambe, T. W., 1958, "The Structure of Compacted Clay", *Journal of the Soil Mechanics and Foundations Division, ASCE*, SM2.
20. Lambe, T. W., and Whitman, R. V., 1969, "Soil Mechanics, SI Version", John Wiley and Sons, Inc.
21. Lee, K. L., Morrison, R. A., and Haley, S. C., 1969, "A Note on the Pore Pressure Parameter B", *Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico*, Vol. 1, pp. 231-238.

22. Matrecon, Inc., 1980, "Lining of Waste Impoundment and Disposal Facilities", EPA SW-870.
23. Mesri, G., and Olson, R. E., 1971, "Mechanisms Controlling the Permeability of Clays", *Clays and Clay Minerals*, Vol. 19, pp. 151-158.
24. Mitchell, J. K., 1976, "Fundamentals of Soil Behavior", John Wiley and Sons, Inc.
25. Mitchell, J. K., Hooper, D. R., and Campanella, R. G., 1965, "Permeability of Compacted Clay", *Journal of the Soil Mechanics and Foundations Division, ASCE*, Vol. 91, No. SM4, pp. 41-65.
26. Mitchell, J. K., and Younger, J. S., 1967, "Abnormalities in Hydraulic Flow Through Fine-Grained Soil", *Permeability and Capillarity of Soils*, ASTM STP No. 417, pp. 106-139.
27. Murry, H. H., 1951, "The Structure of Kaolinite and its Relation to Acid Treatment", Ph.D. Thesis, University of Illinois, Urbana, Ill.
28. Nebergall, W. H., Schmidt, F. C., and Holtzclaw, H. F., 1976, "General Chemistry", 5th ed., D. C. Heath and Company.
29. Olson, R. E., and Daniel, D. E., 1981, "Measurement of the Hydraulic Conductivity of Fine-Grained Soils", *Permeability and Groundwater Contaminant Transport*, ASTM STP 746, Zimmie and Riggs, Eds., American Society For Testing and Materials, pp. 18-64.
30. Skempton, A. W., 1954, "The Pore Pressure Parameters A and B", *Geotechnique*, Vol. 4, pp. 143-147.
31. Sridharan, A., and Venkatappa Rao, G., 1973, "Mechanisms Controlling Volume Change of Saturated Clays and The Role of the Effective Stress Concept", *Geotechnique*, Vol. 23., No. 3, pp. 359-382.
32. Toon, E. R., Ellis, G. L., And Brodkin, J., 1968, "Foundations of Chemistry", 1st Ed., Holt, Rinehart, and Winston, Inc.
33. United States Army Corps of Engineers, 1980, "Laboratory Soils Testing", *Engineering Manual EM 116-2-1906*.

34. United States Department of Interior, 1974, "Earth Manual", Bureau of Reclamation, Denver, Colorado.
35. Williams, N. D., 1982, "The Effects of Elevated Temperature on the Engineering Properties of Seafloor Sediments", Ph.D. Thesis, University of California, Berkley.
36. Zimmie, T. F., Doynow, J. S., and Wardell, J. T., 1981, "Permeability Testing of Soils for Hazardous Waste Disposal Sites", Proceedings of the Tenth International Conference on Soil Mechanics and Foundations Engineering, Stockholm, Sweden.